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## TRANSACTIONS.

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514.

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### LONGITUDINALS vs. CROSS-TIES FOR RAILWAY TRACKS.

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By E. E. RUSSELL TRATMAN, Assoc. M. Am. Soc. C. E.

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#### WITH DISCUSSION.

The following remarks were suggested by the announcement of the title of the paper by Mr. T. C. Clarke on "The Advantages of a Longitudinal Bearing System for Railway Tracks," but those dealing with the subject generally, were in the main originally prepared before the writer had seen the paper, and when he did not know, therefore, that Mr. Clarke would confine his attention to metal longitudinals. The remarks referring specially to points in his paper were, of course, written after a careful perusal of that paper. The general opinions of railway engineers in countries where the continuous bearings of longitudinals and the separate bearings of cross-ties have both been tried, appear to be decidedly in favor of the cross-tie system for ordinary purposes, although the longitudinals continue to be quite extensively used under special conditions, more especially across bridges.

The question of the use of wooden longitudinals presents some points of interest, and will be considered first. In England, the Great Western, the London and Northwestern, the Lancashire and Yorkshire, the

Midland, and other railways, at one time used longitudinal timbers, but the system has been generally abandoned, except in some cases for bridge tracks. The Great Western Railway still maintains the longitudinal system upon its main line of broad and mixed gauge between London and the west of England, and upon the standard gauge line between Swindon and Gloucester, but does not use it on other parts of its lines, or construct any new track of this type. The rails are of bridge section, weighing 68 pounds per yard, and are secured by fang bolts which pass through the flanges. The rails do not rest directly upon the longitudinals, but upon a continuous line of yellow pine shims about 6 inches square and 1 inch thick. The longitudinals are creosoted timbers 15 inches wide,  $7\frac{1}{2}$  inches deep on the outer side and 7 inches deep on the inner side. They are connected by transverse timbers about 6 x 6 inches at intervals of 11 or 12 feet. This is a remarkably easy riding road, especially in the modern cars on four-wheel trucks.\* The broad gauge lines are to be converted to standard gauge in May, 1892. Mr. Lambert, the General Manager of this railway, has informed the author that wherever it becomes necessary to relay any portion of the old broad gauge track the new standard pattern of bull-head rails is being adopted, but there is no intention of abandoning the old longitudinal system when the gauge is altered, where the track is in good condition and does not require renewal. Of course, gradually, as the old track becomes worn out, it will be replaced by the modern pattern;† but it will be a long time before the whole of it has disappeared from the Great Western Railway system.

The comparative costs, at approximate English prices, per mile of single track, including tracklaying but exclusive of ballast, for longitudinal and cross-tie systems, are about as follows:

Great Western Railway.—Bridge rails on longitudinal timbers .....	\$9 440 00
London and Northwestern Railway.—Bull-head rails in cast iron chairs on wooden cross-ties.....	10 475 00
West Riding, Hull and Grimsby Railway (before acquired and relaid by Great Northern Railway).—	
Flange or tee rails on wooden cross-ties.....	8 640 00

\* This track was described and illustrated by the author in *Engineering News*, New York, February 21st, 1891.

† Described and illustrated in the author's paper on "English Railway Track," *Transactions American Society of Civil Engineers*, June, 1888.

At a meeting of the Institution of Permanent Way Inspectors, England, in 1887, it was stated that several advantages were gained by the use of the longitudinal bearing. The rails do not have to act as girders, and may therefore be very considerably reduced in weight, and being generally of the bridge or flange sections, cast-iron chairs are not required. It is safer in case of derailment, as the wheels may run, and have run along the longitudinals until the train has been stopped. On the other hand, the larger scantling of timber required makes it more expensive per foot, and more awkward in repairs, while to replace one timber the whole rail length must be taken up. The trackmen must have some practical and technical knowledge of carpentry as well as of ordinary track-laying, and the work is rather complicated, especially at frogs and switches. It is difficult to keep the drainage perfect, as it is confined between the longitudinals. In a report on a derailment accident on the Great Western Railway, in 1889, one of the inspectors of the Board of Trade stated that it is difficult for the trackmen to know if longitudinals are thoroughly tamped until they watch an engine passing over the places that have been under repair, as the stiffness of the rail and the longitudinal keeps the latter level, even when it may not be properly packed underneath with ballast.

The following are the quantities of timber per mile of single track on different classes of track:

	Feet, B. M.
(A.) English; cross-ties 9 feet x 10 x 5 inches, uniform spacing of 3 feet, 10 ties to a 30-foot rail = 1 760 ties per mile.....	66 000
(B.) English; cross-ties 9 feet x 10 x 5 inches, closer spacing at joints, 12 ties to a 30-foot rail = 2 112 ties per mile.....	79 200
(C.) American; Boston and Albany Railroad, cross-ties 8 feet x 7 x 8 inches, uniform spacing 2 feet center to center, 2 640 ties per mile.....	98 560
(D.) American; cross-ties 8 feet x 6 x 8 inches, 2 800 ties per mile.....	89 600
(E.) English; Great Western Railway, 7 feet gauge, longitudinals 15 x 7½ x 7 inches, with 480 transoms 6 feet x 6 x 6 inches, at intervals of 11 feet.....	104 340
(F.) Suggested plan; longitudinals 12 x 7 inches, with 352 transoms 49 x 6 x 6 inches at intervals of 15 feet....	78 232

The latter plan represents the longitudinal system reduced to a

minimum, and rails with a base less than 6 inches wide would probably cut and split the timber unless shims or tie-plates were used.

In nearly all the writings on this subject great stress is laid upon the difficulty of effecting proper drainage of the roadbed, but it seems to the author that this difficulty is made to appear too great. It is mainly a question of ballast, and there would probably be considerable trouble with sand, earth, or other close ballast. But with broken stone, or even coarse gravel, if free from clay, there should be much less trouble, although the latter might certainly tend to pack into a hard mass to a certain depth. With longitudinals laid upon a bottom course of large broken stone, and well ballasted and packed with broken stone of the ordinary size, there should be ample drainage, even for a sudden heavy rain-fall.

The use of longitudinal timbers to carry the rails across bridge floors is quite extensively adopted in other countries. The two tracks across the great Forth Bridge, Scotland, are laid with rails of bridge section weighing 120 pounds per yard, and secured to teak longitudinals by screw spikes 8 inches long, which pass through the flanges of the rail. There are 20 of these spikes to each rail. The longitudinals are about 12 x 6 inches in section, and are kept in line by horizontal and vertical wedges driven between them and the sides of the iron "trough" in which each line of rails lies, and which prevents the use of transoms or other transverse connections to maintain the gauge. The timbers rest upon a continuous bearing of wooden blocks with pitch filling. There is no ballast. The track and floor seem to be too rigid, needing some means of absorbing the vibrations. The trains make a harsh metallic sound and a jarring in running across the bridge, and these are unpleasantly noticeable in riding in the cars. The change is very pronounced when the train runs onto the embankment approach, where the track is laid with the same rails on longitudinal timbers, with transoms, in broken stone ballast, the train then riding smoothly and quietly. The Severn Bridge, England, is a single track structure, 3 385 feet long, with 21 through spans. The track is laid with rails of bridge section secured by fang bolts and screws to creosoted pine longitudinals, 14 x 7 inches in section, connected at intervals of 11 feet by timber transoms. Guard rails of angle iron 4½ x 4½ inches are secured to the longitudinals along the inner side of the rails, bearing against the rail flange, and leaving 2 inches clear between the rail and

guard. This track was described in the paper already referred to on "English Railway Track," *Transactions Am. Soc. C. E.*, June, 1888. The London and Northwestern Railway, England, has bridge rails on longitudinal timbers across a viaduct near Stockport. A peculiar method of continuous bearing for rails is that used by the Midland Railway, England, across some short span bridges. Each rail is carried by two continuous angle bars, bolted to the rail and to the longitudinal timbers upon which they rest.

It is a very general practice on English railways to lay longitudinal timbers across bridges, generally without ballast, but these do not give the rail a continuous bearing, as the cast iron chairs carry the bull-head rails clear of the timbers. This kind of track, consisting of bull-head rails in cast iron chairs secured to longitudinal timbers laid on a solid floor of transverse iron troughs, is to be used for the elevated electric railway at Liverpool, England. This railway is a plate girder structure, with through spans, and a floor of transverse troughs, and the longitudinals will be kept in position by brackets riveted to the tops of the troughs. In India and the colonies, however, where flange rails are extensively used, it is a very general practice of the English engineers to lay these rails on longitudinal timbers across bridges. Longitudinal timbers across bridges are also used on railways in France and other European countries, the tracks consisting sometimes of double-headed rails in chairs, and sometimes of flange rails resting on the longitudinals and secured by bolts or screw spikes passing through the flanges or merely bearing upon them. Angle iron brackets riveted to the bridge floor may be used to keep the longitudinals in position. The Victoria tubular bridge which carries the Grand Trunk Railway across the St. Lawrence River at Montreal, has a single track laid with flange rails spiked to longitudinal timbers which rest on the transverse I-beams of the floor. Mr. Hannaford, M. Am. Soc. C. E., Chief Engineer, states that the spikes have reversed points. The question of the relative merits of longitudinals or cross-ties for carrying the rails across bridges was discussed at the International Railway Congress held at Paris in 1889. The conclusion arrived at was that both systems possessed certain advantages and were used according to local conditions, and that neither could be definitely recommended to the exclusion of the other. In Fig. 1 is shown the floor system of the double track bridge carrying the Ceinture Railway (or Belt Railway) of Paris across the

River Marne at Champigny. The longitudinals are held down at intervals by straps under the rails and vertical bolts. The floor is covered with ballast. In Fig. 2 is shown the floor system of the single track Gagnières Viaduct, France, in which the longitudinals are laid against angle iron brackets riveted to the floor beams. These beams are below the top chords of the main girders, which thus form a high guard-rail.

In regard to metal longitudinals, the most extensive experiments with these has been in Germany and Austria; but while the use of metal cross-ties is being extended considerably, the longitudinals are only used to a limited extent on new work, and in some places they are being aban-

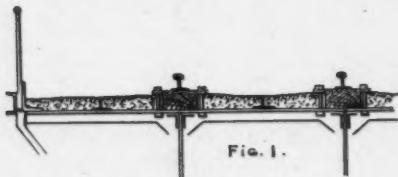


FIG. 1.

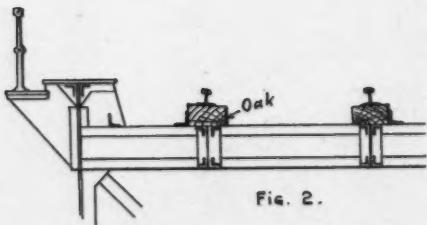


FIG. 2.

doned. The transverse connections generally give some trouble, especially at curves, and the general construction of the roadbed and the work of tracklaying require more care and labor, while the cost of maintenance has in some cases been found to be higher with longitudinal than with cross-ties. There has also been difficulty in maintaining the gauge. A German authority stated in an article in a technical magazine, in 1886, that with longitudinals of metal it is undesirable to have many transverse connections, as they will counteract the advantages of the longitudinals and introduce some of the disadvantages of the cross-ties. He recommended a connection at the ends of the longitudinals, and also at the middle if more were considered necessary. The continuous

bearing is theoretically the better, giving a uniform transmission of the wave motion under rolling loads, with a consequent smoother rolling and less wear. The separate bearings of cross-ties exert a disturbing influence on this wave motion, causing uneven wear, but it is possible to obtain sufficiently satisfactory results in practice in this respect from the cross-tie system. The continuous bearing gives a better resistance to lateral strains on the rails, but with cross-ties this can be provided for by the use of tie-plates. The quantity of ballast is about the same, but there is difficulty in draining it when longitudinals are used. Renewals are probably cheaper with cross-ties. After considering the various points he came to the conclusion that the cross-tie system will continue to be the standard. The same conclusion was also arrived at by a convention of the German Railway Union a few years ago.

Mr. Clarke, in his paper, does not seem to have done his subject justice in merely quoting the statements of Mr. Hohenegger concerning the latter's system of track on the Northwestern Railway, Austria, of which he is Chief Engineer. In the report of the author to the U. S. Department of Agriculture upon "The Substitution of Metal for Wood for Railway Ties" (1890), there are many other statements concerning metal longitudinals and the experience therewith, which show different views from those entertained by Mr. Hohenegger. A longitudinal system would require more care and more skilled labor in laying and maintenance than it is now usual to find on any but the first-class lines in this country, and the work would be especially complicated at frogs and switches and on sharp curves. A defect of this system of track is the very much smaller area of bearing surface upon the ballast as compared with that of the cross-tie system, and, as already stated, it is not considered desirable to increase the number of transverse connections. This, added to the increased difficulty in tamping, and other features, would tend to add largely to the maintenance expenses by continual work in keeping the track up to surface. Under heavy traffic it would probably not be found advisable to make any great reduction in the weight of the rail. For existing roads adopting this system the entire reconstruction of the track would be a difficult and costly work, especially upon lines with heavy traffic, and it is a question whether the efficiency and economy resulting from the new track would be sufficient to warrant the change. It is only upon lines with heavy traffic that the introduction of the longitudinal system of track is likely to be con-

sidered, even if its advantages are proved; but the traffic upon the lines mentioned by Mr. Clarke is comparatively light.

The following extract from the report just mentioned is the summary of the information therein presented in regard to longitudinal bearings, and in regard to the Berlin Metropolitan Railway, later information is to the effect that after six years of comparatively heavy traffic the iron longitudinals were abandoned for wooden cross-ties on account of the insufficient support across the track for bearing surface and for resistance to lateral strains upon the track:

**LONGITUDINALS.**—This type of track is only used to a limited extent, and, as shown by special reports and other information, its use is not increasing. The construction of the track is more difficult and requires more labor than that with cross-ties. Maintenance, renewals and repairs are less easily managed, and greater care must be paid to the ballasting and drainage. On the other hand, it has the advantage of giving the rails a continuous support, and, consequently, with a well-packed roadbed would make a very smooth riding track, but probably at a higher cost than an equally good track of another type. The construction of the roadbed and arrangement of the ballast involve considerable care and cost, as, owing to the difficulty of drainage, special means have to be taken, by the use of courses of large rough stone or drain pipes, to carry water away quickly. Longitudinals have been adopted for the city railways of Berlin, Germany, having been proved the best for reducing the noise of passing trains on this viaduct line connecting main lines of railway. It was originally thought that longitudinals would make a better and cheaper track, avoid shocks at the rail joints, and, by their long bearing in the ballast, require less work of maintenance. Also, that lighter rails could be used with the continuous bearing. It is, however, difficult to maintain the ballast so as to keep a continuous and even bearing inside the longitudinal. The economy in material effected by the use of longitudinals is reduced or negatived by the necessity of using transverse connections of ties or tie-rods to hold the track together and maintain the gauge. This system is awkward on curves, and renewals are difficult. Each piece must be bent hot at the works, or have the holes for the rail fastenings made to fit a certain radius of curve.

In regard to steel cross-ties, Mr. Clarke says: "The tie being but a shallow piece of channel iron with flanges 2 to 3½ inches deep has no stiffness crosswise." This seems to imply that there is but one form of cross-tie, but in point of fact they are of various forms and sections. The channel or trough section, of certain shapes, has been found in general the most satisfactory, and it is not difficult to stiffen the ties if found necessary. Many of these now in use, however, are sufficiently stiff to retain their shape under heavy traffic, and give good satisfaction. Mr. Clarke's remarks upon the necessity of secure fastenings, apply equally to fastenings for cross-ties.

On the whole, it appears to the author probable that the cost of a system of track with longitudinals, fitted for heavy traffic, would be considerably in excess of the cost of a track with cross-ties of equal strength and efficiency. It is true that some engineers are in favor of the longitudinal system, theoretically at least, and it is possible that it may constitute the track of the railway of the future; but its introduction does not seem probable, and the most important work now to be done is to provide for the improvement of the efficiency of the present system of track. These remarks apply to the track in general, but there may be special cases, as at bridges, etc., where the use of longitudinal bearings may be advantageous.

Granting that much of the existing track is defective, the author does not think there is room for claiming that (as stated in Mr. Clarke's paper) the system of track is giving out at all points, for the existing tracks are capable of very great improvement without departing from the present general system of construction.\* The adoption of heavier rails is continuing, but the extra cost deters many companies from relaying their track even when it is admittedly too light for the traffic. In addition to the heavy rails mentioned in the paper above referred to, the single track of the St. Clair tunnel at Port Huron, Mich., connecting the Grand Trunk Railway and the Chicago and Grand Trunk Railway, is laid with rails of the Sandberg pattern, weighing 100 pounds per yard; the Boston and Albany Railroad has adopted a 95-pound rail; the Manhattan Railroad (New York) and the Chicago and South Side Rapid Transit Railroad (both elevated railways) are using 90-pound rails; the Baltimore and Ohio Railroad is laying 85-pound rails upon its Philadelphia and Baltimore Division; the Great Northern Railway has laid 80-pound rails on the heavy grades of the Pacific extension of the St. Paul, Minneapolis and Manitoba Railway, and the New York, Lake Erie and Western Railroad and the Lehigh Valley Railroad have adopted 80-pound rail sections. It may be noted also that the government of South Australia has adopted an 80-pound flange or tee rail for use on the main lines of its railways.

It is to the interests of the railways to employ skilled labor or at least intelligent labor on all work of importance, as cheap labor is dear labor in many cases. An instance of this came under the author's notice at

\* This view was presented in his paper on "The Improvement of Railway and Street Railway Track," *Transactions American Society of Civil Engineers*, March, 1890.

Montreal last summer while watching the loading of new steel rails upon Grand Trunk Railway flat cars. The rails were in a triangular pile, and half a dozen men were engaged in lifting them by hand and throwing them on a truck or cart so that they struck forcibly against one another, sometimes falling on the head and sometimes on the side. When all the loose rails at the bottom of the pile had been thus loaded on the truck, the top of the pile was pushed over, about a dozen rails rolling over and over, and writhing and bending like snakes as they struck the ground. When the loading was completed the truck was driven around to the side of a flat car, and being of the same height, two iron bars were placed across as skids. The rails were then thrown onto the skids, striking on head, side or base, then slid across, lifted with rail tongs and placed or dropped upon the other rails on the car. Supposing similar methods to have been practiced in loading at the mills and into the ship, unloading from the ship and again in unloading on the track, it may readily be imagined that many of the rails were subjected to serious injury by this carelessness in handling.

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## DISCUSSION.

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T. C. CLARKE, M. Am. Soc. C. E., writes: Mr. Tratman brings up evidence to show that in the general opinion of railway engineers and managers, the longitudinal system gives a smooth, easy riding track, but that it is more expensive to construct and maintain than a cross-tie system. When both systems are of wood there is no doubt of the truth of this conclusion.

The fourteen years' experience of Herr Hohenegger with a metallic longitudinal system does not seem to show that that system is more expensive to construct or maintain than a metallic cross-tie system. My own views can be briefly stated. So long as we can get wooden cross-ties at any reasonable cost we should continue to use them and endeavor to remedy their defects. When their cost becomes prohibitory, we should consider more carefully as between metallic cross-ties and metallic longitudinals.

The defects of the present system are: 1st, decay of the ties. This can be remedied by preservative processes. 2d, Spreading of rails. This can be prevented by some form of chair or tie-plate held to the tie by four spikes. This also prevents a third defect—cutting of the rail into ties. The fourth defect is, in my opinion, not to be remedied by the

present joints of rails. Two discontinuous rails cannot be united by any form of splice which will give a perfect joint under our heavy and rapid trains. Either the connection will be loose and deflection will take place, or if tight enough to prevent that, expansion, not being able to force the rail ends together, will cause kinks in the track, as was seen on the Lake Shore Railway. We want to return to a compound rail, made in two parts breaking joints, the parts united by bolts and rivets in larger holes. The office of these bolts or rivets should be not to transmit shear, but to hold the two parts of the rail together, and the shear will be transmitted by one part sliding upon the other. Such a rail, being of equal strength and elasticity throughout, would practically be jointless, whether it rested on ties or on longitudinal bearers.

When the time comes to decide between longitudinals or cross-ties of metal, the question becomes one of giving enough bearing surface, for with either system we prevent decay and spreading of rails, and by the use of compound rails can also abolish low and battered joints.

The trouble with all metallic cross-ties—either those illustrated by Mr. Tratman in his Government report, or those that have been since invented—is, that from motives of economy the designers have not given them enough bearing surface. They are all insufficient in depth to transfer the load on the rails to their centers. Hence the whole length of the tie cannot be available for bearing. A wood cross-tie system, with 8-foot ties of 8 inches face, 2 feet between centers, gives 1.32 square feet of bearing per linear foot of rail, and this is the least allowed for our present loads, for the reasons so clearly pointed out by Dr. Dudley and Mr. Shinn. None of the metallic cross-ties in use give much over one-half a foot of bearing surface per linear foot of rail.

The merit of the longitudinal bearing system is that the bearing is all directly under the load, and where it does the most good.

J. FOSTER CROWELL, M. Am. Soc. C. E.—Mr. Tratman has given us a great deal of information in this interesting paper as to the use in the past of longitudinal track supports, and the facts he cites are instructive in that they show what types have failed and why their undoubted advantages for certain purposes have been overbalanced. In regard to what may be termed the modern revival of the longitudinal system, however, he does not give us much detail, but contents himself with a résumé or summary of general information; he leaves the question open as to whether or not it is practicable to devise a longitudinal system that shall retain the acknowledged advantage of continuous bearing, and yet be free from the objections which have practically driven it out of the field, generally speaking.

In considering the relative uses of the two systems we must not lose sight of the fact that the cross-tie track has been retained not on account of excellence but in spite of almost fatal defects, to remedy which an enormous concentration of thought and effort extending over many

years has been expended without removing them. It can only be regarded, then, as something to be tolerated until we can be supplied with something better.

We are, most of us, inclined, I presume, to share Mr. Tratman's conclusion that the introduction of the longitudinal system, in a radical and general sense, now seems improbable; and yet it is difficult to state precisely any theoretical or practical objection to its use that cannot be offset by one against the other system, save two, which are, the greater first cost, and the greater time and effort required to make it effective and keep it so. The first is in itself no argument, though a potent factor, but the second is most important both as factor and argument. We can, in other words, readily secure with the cross-tie in all situations, on all road-beds, and in fact, in emergencies, even without a road-bed, what would be difficult and often impossible with the longitudinal, viz., immediate working efficiency; and this whether we are dealing with new track recently laid or old track which has been washed out or otherwise impaired, or on a road-bed being ballasted in service, as so frequently happens, in this country at least. This is such a vital consideration that in most cases it would govern even if the two systems were otherwise balanced in merit and of equal cost; but its potency diminishes as the conditions of permanency in road-bed and ballast are attained.

Many will agree, probably, that there is good ground for concluding that though the longitudinal is not likely to supplant the cross-tie, it may, before long, be used to supplement it in special cases, such as, for instance, on lines crowded to such an extent with heavy high speed traffic that permanence of way comes to represent not merely a theoretical but a direct and obvious money value, worth saving; also in some tunnels wherein renewals and maintenance are unusually difficult and obstructive; also on specially built urban lines, in terminal stations, and on approaches where direct economy is found in perfection of service or where disturbance of the road-bed is not permissible. Already we see illustrations of the latter applications in the costly cable road constructions (parenthetically these show us, too, that rigidity of track is conducive to smoothness of motion and quiet).

Mr. Tratman places, I think, rather undue stress on the difficulty and cost of substituting a new system on an established railroad in operation, although they are undoubtedly of sufficient importance to deter experimental efforts in that direction. On the other hand, we see that the aggregate cost, eventually, of the cross-tie track, after incorporating adequately the various improvements which Mr. Tratman enumerates as being necessary to secure efficiency; will be brought up to such a figure that the financial argument against introducing the longitudinal features will be greatly weakened, especially if by that time the economy of the additional expenditures shall have been practically proved by increased service.

Whether a track can be evolved, all parts of which can be kept permanently in place and in full efficiency, excepting the top member that wears directly under the wheel, and wherein even the wearing part is compacted and not ruptured by the stresses, is still almost as much as ever a matter of conjecture; but it is at least safe to say that, considered with reference to the present and growing conditions of railroad service, such a theoretically perfect structure would not be either a longitudinal or a cross-tie track, but would be both; that is, it must combine in one all the advantages of each.

E. R. TRATMAN, Assoc. Am. Soc. C. E.—In reply to Mr. Crowell's discussion, it may be said that the paper refers more especially to the ordinary open track under ordinary conditions, than to exceptional conditions or circumstances, but at the same time states that there is a possibility of the introduction of the longitudinal system at certain localities and under such exceptional conditions, as at bridges, etc. In this connection it may be of interest to note that the tracks in the new Union Station (Louisville and Nashville Railroad), at Louisville, Ky., consist of 80-pound rails laid on continuous iron plates upon longitudinal timbers 12 x 12 inches in section, which rest on wooden cross-ties. The spikes pass through the iron plates. The ballast filling is brought up level with the tops of the longitudinals. Mr. Crowell's suggested combination of longitudinals and cross-ties, presumably similar to the tracks just mentioned, is a development of the old arrangement very generally used for street railways. Such a construction for a railway, however, would require a very large amount of timber. Where heavy steel ties have been used under the joints of steel longitudinals on foreign railways, it has in some cases been found necessary to replace them with lighter connections. It is possible, however, that a complete track on this combination system might be designed, built entirely of steel and having efficient fastenings, which would present advantages of such force as to lead to its trial on busy sections of track with heavy and constant traffic. It would be an expensive construction, but if successful might lead to a considerable reduction in maintenance and repairs.

What the track of the future will be it is not easy or safe to predict, but the improvement of railway track, either radically or in detail, is a problem occupying many minds. The author recognizes the advantages of the longitudinal system, as has already been shown; but its general introduction on the ordinary open track does not seem probable, and under such ordinary conditions of track and traffic it is likely that the present system of track, with cross-ties, can be very much improved at less cost than would be involved in the introduction of a new system.

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NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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515.

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FREE RAILWAY CONSTRUCTION vs. GOVERNMENT CONTROLLED AND OWNED RAILWAYS.

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By E. BATES DORSEY, M. Am. Soc. C. E.

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WITH DISCUSSION.

Under the head of Free Railway Construction will be included all railroads that have been or can be built by individuals or companies, without special acts of Government, whether national or State. Under the head of Government-Owned Railways will be included those owned and operated by the Government; and under the head of Government-Controlled Railways will be included all those that cannot be built without a special act of Government, as is the case with all English railways; in these the English Government has no direct or indirect ownership whatever, but owing to the great difficulty and cost of getting from Parliament special acts to build new lines of railways, the present English roads have a virtual monopoly of the land transportation of the United Kingdom. It is needless to say that the roads make the most of this exclusive right, by charging freight rates that to Americans appear excessive, half of which would fill the hearts of our Railroad Managers with joy.

All that can be said in favor of Government ownership or control of

railways is, that it can regulate the freight charges and prevent excessive rates that would interfere with the industries of the country. Practically it has no such effect. The result of the writer's experience and observation is that the highest railway freight rates in the world are in England and in her colonies, where the railways are either owned or controlled by the Government; and the lowest rates are those in the United States, where there is free railway construction, and consequently great competition, which has obliged our managers to build and operate their railroads upon the most economical plans, and to adopt all new inventions that will reduce the cost of construction or operating expenses. This has made it possible for them to transport freight over inferiorly constructed roads at less than half the English cost.

The following table shows some of the current freight rates in England, South Africa, and the United States:

$\text{£1} = \$4.80.$	Miles of Haul.	FREIGHT CHARGE PER TON PER MILE.		
		England.	South Africa.	United States.
Birmingham to London, hardware.....	113	Cents.	Cents.	Cents.
Birmingham to Manchester, hardware.....	90	a 5.85	.....	.....
Stoke-on-Trent to Liverpool, pig iron.....	55	a 5.73	.....	.....
Birmingham (U. S.) to Pensacola, pig iron.....	280	a 2.32	.....	.....
Birmingham (U. S.) to Pensacola, coal.....	280	.....	.....	b .50
Merthyr (Wales) to London, coal.....	177	.....	.....	b .50
London to Oldham, cotton.....	188	a .995	.....	.....
Liverpool to Oldham, cotton.....	43	a 2.56	.....	.....
Cape Town to Kimberly, grain (domestic).....	647	.....	b 3.	.....
Cape Town to Kimberly, iron.....	647	.....	b 7.	.....
Port Elizabeth to Kimberly, grain (domestic).....	485	.....	b 3.	.....
Port Elizabeth to Kimberly, grain (imported).....	485	.....	b 6.	.....
Cape Town to Kimberly, grain (imported).....	647	.....	b 6.	.....
Port Elizabeth to Kimberly, iron.....	485	.....	b 7.	.....
Durban to Biggarsberg—Natal, grain.....	231	.....	.....	a 8.
Durban to Biggarsberg—Natal, iron.....	231	.....	.....	a 9.25
Newcastle to Durban—Natal, coal.....	270	.....	.....	a 1.50
Average charge for all freight on Pennsylvania Railroad Division. Average haul.....	135	.....	.....	b .626

NOTE.— a = ton of 2240 pounds.

b = ton of 2000 pounds.

The railway from Durban has many grades of one in thirty, and one summit of 5 155 feet above its initial point. The first part of the road is very badly located for cheap working. Coal on the railways from Port Elizabeth and Cape Town costs from \$8 to \$12 per ton. All the railways in South Africa are owned and operated by the Colonial

governments; they are very well constructed and maintained, much better than the average of our American roads.

In a former paper read by the author before the Society, and on page 112 of his book "English and American Railroads Compared," the author has compared the London and North Western Railway of England to the Pennsylvania Railway Division of the United States. The comparison is here continued down to the year 1889.

Average cost of transporting one ton or one passenger one mile. £1 = \$4.80.	London and North Western.		Pennsylvania Railroad Division.	
	1884.	1889.	1884.	1889.
Maintenance of way.....	Cents.	Cents.	Cents.	Cents.
Repairs, renewals of locomotives.....	.209	.208	.103	.089
Total cost of motive power.....	.082	.085	.044	.034
Total operating expenses.....	.271	.286	.148	.130
	1.130	1.208	.530	.491

This shows that in the last five years the total operating expenses of the London and North Western has increased 7 per cent., while that of the Pennsylvania has decreased the same percentage.

In comparing American railways with those of Europe, the usual mode of comparison between the cost of train miles is misleading and erroneous, owing to the great difference in the train load of paying freight; which is only 59 tons on the Great Northern of England. The general average of all the railways of the United Kingdom is 71 tons—or less than half of the average paying load of the American trains. It is like comparing the cost of a donkey cart with that of a six-horse wagon.

Much has been said of the great advantage the American railways have over the English in their long freight haul; this argument is entirely false, as the average receipts per ton on the English roads are about the same as those on the American, the only difference being that the American hauls the freight more than three times the distance for about the same money. For example, the average haul of the Pennsylvania Railroad Division is 135 miles, with average receipts per ton of eighty-five cents; on the London and North Western Railway of England, the average haul is about 35 miles, and the average receipt per ton eighty-four cents. The author can see no advantage of the long haul in this case.

The severe competition between railways in the United States is undoubtedly very bad for the shareholders, but it has been the making of the country. Suppose we estimate the sphere of beneficial influence to extend 12 miles each side of the line; this gives 15 360 acres of land for each mile of road; estimate this at \$20 per acre, and it makes the total valuation of the land benefited by the railway \$307 200, while the cost of the mile of railway is, say, \$30 000, or 10 per cent. of the value of the land benefited. The appreciation in the value of the land is very much larger than the cost of the railway, and the investment would pay even if the entire cost of the railway was lost.

The leading English railways pay the following dividends on their ordinary shares.

	Per Cent.	
	1888.	1889.
London and North Western.....	7½	7½
Midland.....	6	6
Great Northern.....	4½	4½
Great Western .....	6½	6½
London and South Western.....	6	6
Taff Vale.....	5	5

This shows that the English railways pay very well when their great cost is considered, but the money is undoubtedly earned at the expense of home industries. This is seen very forcibly in the depreciation of farming lands during the last twenty-five years—since the wheat of our Western States has come into direct competition with the English farmer; notwithstanding that the first has to pay in many cases over 1 500 miles of railway freight and 3 500 miles of freight by sea, while the last has only to pay railway freight for less than 300 miles. In spite of these great disadvantages, the American farmer competes so successfully with the English, that he has depreciated very largely the value of English farms.

If the English freight rates were in force in this district (Eastern Tennessee), the iron and coal interest surrounding us, that is being so extensively worked under our low railway rates, could not exist; and in place of the busy mines and furnaces there would be only a few farmers raising enough food for their own support, shipping, perhaps, a little cotton, for nothing else could stand the heavy railway freight rates to the seaports.

In English South Africa all the railways are owned by the Government, and the evil effects of the state ownership can be readily seen. In order to get the necessary votes in the Assembly to pass a bill authorizing the construction of meritorious lines, it has been necessary to build other lines that were not required and will not pay. To secure the required votes they must make an Omnibus bill, including the different railways proposed; following there in their railway bills the same practice that we do in our river and harbor bills.

As an example of this, the Cape Colony Government railways of South Africa, operating 1 599 miles, which cost £14 282 766, earned in 1889 a net profit equal to a dividend of 5½ per cent. on the total cost. The road from Port Elizabeth to Kimberley, a part of the system, 485 miles long, costing £4 300 000, earned in the same year 13½ per cent. on its total cost. This branch transported large quantities of mining machinery and goods to the interior for the diamond and gold mines. This branch shows the remarkable receipt of over \$17 for each ton of freight handled, which the author believes to be unequalled on any other road.

As the Government and Railway officers wish to make a good financial showing, they are obliged to charge high freight rates on the good lines, to make up for the loss on the poor ones. This has caused such high freight rates as to be nearly prohibitory to farming except near the sea-coast. Diamond and gold mining, sheep and ostrich farming are the only industries developed in the interior that can stand the high freight charges. The Government will not grant permission to construct private lines, for fear of competing with and injuring the public lines.

Many reasons such as republican government, free government, free religion, rich soil, etc., have been given as the cause of the great prosperity of the United States; but the real cause has been the law allowing free railway construction. This has given the country much greater railway facilities than it could have had otherwise, and the sharp competition has given such low freight rates, that the interior of the United States, notwithstanding its thousand miles of railway freight, can compete successfully in Europe in its food products.

On the English railways, which have cost from four to six times more to construct than the American roads, the cost for transporting freight is more than double the American cost, whereas it should be much less, or why make the increased outlay and expense for superior construction?

The author thinks that the great difference in favor of American rail-

#### DORSEY ON FREE RAILWAY CONSTRUCTION.

ways in the cost of operating expenses is owing principally to the following reasons:

*First.*—The trains on the American railways carry much larger loads than those on the English roads.

*Second.*—The universal use on the American railways of rolling stock with bogie-trucks which run with much less friction and wear and tear than the English rolling stock, with its long, rigid wheel base.

*Third.*—The general use on the American railways of freight cars or wagons carrying a greater percentage of paying load to the dead weight than those used on the English railways.

*Fourth.*—The lower speed at which the American goods or freight trains run.

*Fifth.*—The use on the American railways of heavier locomotives, which haul heavier loads than can be done by the lighter ones used on the English railways.

*Sixth.*—The use of locomotives on the American railways with outside cylinders and connections, which can be more cheaply and easily repaired than those with inside cylinders used on the English railways.

*Seventh.*—The use on the American railways of collecting and distributing freight trains which load and unload at stations freight in less than car load quantities, thus avoiding leaving cars only partially loaded.

The best authority gives the average load of the English freight car at from  $1\frac{1}{2}$  to  $2\frac{1}{2}$  tons, while the weight of the wagon will average over 5 tons—making the tare more than double the paying load. As soon as the English Railway managers are convinced, and will act upon this conviction, that it costs as much to haul 1 ton of tare or dead weight 1 mile as it does 1 ton of paying load, they will then make a very large saving in the cost of transportation, by reducing the present excessive tare.

If there was free railway construction in the United Kingdom, with new roads built and run also, according to modern railway practice, these would by competition force the old roads to improve their rolling stock, and consequently reduce largely their operating expenses, thus enabling them to make large reductions in their freight charges, without interfering with their dividends.

Those who are working for State interference, control, or ownership of railways, in order to reduce the freight charges, should travel and see what is being done in other countries, where the railroads are more or

less controlled by the Government; they would return convinced that railway facilities and freight charges in the United States, under our free railway construction laws, are by far the best and the lowest in the world. They should look into the financial condition of the offending road, when, if they can show large or excessive profits, it would be very easy to obtain capital to build competing lines—promoters and contractors being constantly on the lookout for such opportunities. The most dissatisfied Granger cannot say that our railways are now paying unreasonably large dividends.

The author, after long and close observation in other countries, is of the firm conviction that the best way to reduce freight rates is to have free railway construction and operation and its consequent competition; as it will give cheaper rates and greater facilities than can be had under Government-controlled or owned railways and Government management.

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## DISCUSSION.

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ROBERT MOORE, M. Am. Soc. C. E.—I think this paper might very well be circulated as a tract among our Western legislators. It shows very completely that the people at large have no reason to ask for State ownership of railroads. But the showing for the railroads themselves is not so good, and it is a question whether the State might not properly interfere to check somewhat the unlimited construction of railroads which prevails in our country now. There have been under our present system of free railroad construction cases which might almost be called blackmail, of which the benefit either to the community or to the owners of the road is extremely doubtful. So that if a method can be found by which the construction of unnecessary railroads can be avoided, I think that the whole community will be the gainer. It is worth considering whether the partial return to the English practice which is now in vogue in the State of Massachusetts is not advisable. By this system a charter under the general statute is granted only when the incorporators can prove to the Railroad Commission that the road which they seek to construct will be a public benefit, and that they are able to build it. Failing in this, they must go to the Legislature for a special act of incorporation.

Something like this would be a very wholesome check upon over-construction, and may well be commended for adoption in the older States at least.

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### DISCUSSION ON PAPER No. 493.

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By J. C. TRAUTWINE, Jr., EDWIN R. KELLER, F. COLLINGWOOD, G. W.  
PLYMPTON.

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J. C. TRAUTWINE, Assoc. Am. Soc. C. E.—In my discussion of Mr. Plympton's paper on the strength of plate glass, I remarked that he left us without the connecting link between his experiment on bars and their application to plates, and his reply hardly answers the question. I found that Grashof gives formulas for the strength of plates with their edges confined; and on application to Edwin R. Keller, of the University of Pennsylvania, he furnished the following discussion:

EDWIN R. KELLER, M. Am. Soc. C. E.—The experiments of Professor Plympton on the transverse strength of Dutch plate glass, were not performed under the conditions to which glass plates are subjected in actual practice; yet we may, by the following method, which is similar to that adopted by Weisbach in the discussion of flat boiler sheets, formulate the results in such a way that they will obtain approximately for the case of a rectangular plate fixed in water-tight fastenings and uniformly loaded.

Consider the plate to be made up of a number of strips such as those used in the tests, and suppose them to be placed with the length ( $a$ ) horizontal. Suppose a strip of length  $a$  and width unity to carry per

linear inch the portion  $w_1$  of the unit pressure  $w$ . Then if  $R$  is the unit stress in the most strained fiber and  $d$  the thickness, we have:

$$\frac{1}{12} a^2 w_1 = \frac{R d^3}{6}$$

$$\text{and } d = a \sqrt{\frac{w_1}{2 R}}.$$

The max. deflection occurs at the center and is given by

$$\delta_1 = \frac{1}{384} \frac{12 w_1 a^4}{d^3 E}.$$

Similarly if we consider the plate made up of a number of strips of length  $b$ , placed vertically and carrying the portion  $w_2$  of the pressure  $w_1$  on each linear inch, we obtain:

$$d = b \sqrt{\frac{w_2}{2 R}} \quad \text{and} \quad \delta_2 = \frac{1}{384} \frac{12 w_2 b^4}{d^3 E}.$$

Now, evidently, the deflections  $\delta_1$  and  $\delta_2$  must be equal, and therefore,

$$\frac{\delta_1}{\delta_2} = 1 \frac{w_1 a^4}{w_2 b^4} \quad \text{and} \quad w_2 = \left(\frac{a}{b}\right)^4 w_1$$

$$w = w_1 + w_2 = \left(1 + \frac{a^4}{b^4}\right) w_1$$

$$w_1 = \frac{w b^4}{a^4 + b^4} \quad w_2 = \frac{w a^4}{a^4 + b^4}$$

$$\therefore d = a \sqrt{\frac{w b^4}{2 R (a^4 + b^4)}} = b \sqrt{\frac{w a^4}{2 R (a^4 + b^4)}}$$

And, in general,

$$d = a^2 b \sqrt{\frac{w}{2 R (a^4 + b^4)}} \quad \text{where } a \text{ is the larger side}$$

of the rectangle.

Considering now the experimental results, we have, for a beam supported at both ends and loaded in the center:

$$\frac{1}{4} Wl = \frac{R b d^3}{6} \quad \text{or} \quad R = \frac{3}{2} \frac{Wl}{b d^3}$$

and, substituting the values obtained in the time tests, we obtain:

Experiment.	R.
No. 4 .....	3 609
8 .....	3 955
9 .....	4 500
Mean .....	4 020

Using this value in the formula, we obtain, for the proper thickness:

$$d = \frac{a^2 b}{89.7} \sqrt{\frac{w}{a^4 + b^4}}$$

$a$  being the larger and  $b$  the smaller dimension of the pane in inches, and  $w$  the wind pressure in pounds per square inch.

GEORGE W. PLYMPTON, M. Am. Soc. C. E.—I am inclined to the opinion that Grashof's method as given by Lanza would afford a more satisfactory approximation than that given by Mr. Keller. The significance of  $w_1$  and  $w_2$  needs to be more clearly stated by Mr. Keller. As the text reads, either of them represents the pressure on a unit area.

F. COLLINGWOOD.—There is one suggestion which occurs to me, viz., whether plate glass of any size could be considered as fixed at the edges. The hold in any case is usually quite narrow, and I very much doubt whether the glass is strictly in the condition called "fixed."

Mr. KELLER.—It seems to me that the significance of the quantities  $w$ ,  $w_1$  and  $w_2$  is stated as clearly as may be; they are not equal to each other, but  $w_1$  is that portion of  $w$  which is carried by a longitudinal strip and  $w_2$  is that portion of  $w$  which is carried by a transverse strip, so that  $w = w_1 + w_2$ , as given in the demonstration.

Referring to Mr. Collingwood's suggestion that a glass plate "is not strictly in the condition called fixed," because the hold is narrow, I would say that since a beam is fixed when the tangent to the elastic curve at the supports is horizontal, it appears to me that if the hold is sufficiently rigid the plate will be fixed, no matter how narrow it is. Rupture is rarely caused in a glass plate by the stress which a given load would theoretically produce, but generally by the inequality in the support of the ends. It is therefore impossible to make any but the roughest calculations. The formula is given only as a rough approximation and as such must be used with an ample factor of safety.

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### EXPERIMENTAL DETERMINATION OF THE ROLLING FRICTION IN OPERATING THE DRAW OF THE THAMES RIVER BRIDGE, TOGETHER WITH METHOD FOR DETERMINING POWER TO OPERATE DRAW-BRIDGES.

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By ALFRED P. BOLLER, Jr., and H. J. SCHUMACHER.

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#### WITH DISCUSSION.

The following paper on the frictional resistance of draw-bridges is submitted to supplement the very meager data on the subject. The only recorded experiments the writers have been able to discover are those published in the *Transactions Am. Soc. C. E.* some twenty years ago, in a paper by C. Shaler Smith on the general subject of draw-bridges. These experiments were made by simple dynamometer measurements, and took no account of the moment of inertia of the structures experimented upon. Further, they were made on comparatively small bridges, and on a quality of workmanship far below what is now within shop possibilities. A careful experimental record, with the deductions therefrom, upon a large modern draw-bridge, was thought to be of sufficient value to be made a matter of permanent record, and yielding useful working data in a field in which little work has been done.

The Thames River Bridge at New London, Conn., was selected, as it represents the greatest achievement thus far in draw-bridge building. It is 500 feet long, weighs 2 400 000 pounds, and is the largest double-track draw in the world. The management of the N. Y. P. & B. R. R. gave all assistance necessary, otherwise the experiments could not have been undertaken. The bridge was opened in the fall of 1889, and had been in successful operation a year and a half when the observations were made from which the results were deduced. The greatest skill and care were exercised in making all the members of the bridge, and particularly in making and setting up the turn-table and track, so as to reduce to a minimum the work of operating due to imperfect workmanship and irregularities.

The following is a description of the turn-table taken from the report by the Chief Engineer :

"The whole weight of the draw is delivered to a rim-bearing turn-table at four points, the supporting cross girders being arranged so as to deliver the weight in exact proportion to eight equidistant points on the circular drum, which, being 32 feet in diameter, divides into 12½ feet segments which measure the extent of the distribution the drum is called upon, to afford a uniform bearing on all the wheels under the influence of each segment. The whole weight rests upon fifty-eight cast-steel 20-inch wheels, with 10-inch face of metal as hard as could be faced in the lathe, weighing 800 pounds each. The estimated weight borne by these wheels is 1 300 tons, or 22 $\frac{1}{2}$  tons per wheel, being 4 400 pounds per lineal inch of face. The wheel treads are rolled steel plates bent to the curve, and faced to a true cone and bearing.

"Extraordinary care was exercised to make the lower tread absolutely level, having all bearings and surfaces as true as machines could make them, and no expense was spared to make the drum rigid and stiff, as the price of a smooth and easy working table. The live ring coupling the wheels is perfectly flexible, being composed of separate band pieces between each pair of wheels. The rack is of cast-iron, with a face of 10 inches, teeth 3 $\frac{1}{2}$  pitch, and a pitch circle of 34 $\frac{1}{2}$  feet in diameter, into which gears two shrouded pinions with thirteen teeth and 15-inch pitch circle, having vertical shafts 5 $\frac{1}{2}$  inches diameter, working in heavy and strongly braced sleeves attached to the drum. The table is provided with both hand and steam power, the former through capstan bars, three for each shaft, engaging into capstan heads attached to the multiplying gear driving the vertical shaft.

"The steam power located within the drum consists of a pair of oscillating cylinders 10 inches in diameter with a 7-inch stroke, on a compact frame setting at an angle of 120 degrees, and working upon one crank shaft; running at a maximum velocity of 200 revolutions (average about 170), applying the power through two Frisbie friction clutches, the one driving the unlocking shaft, the other the turning shaft. The turning shaft, of 4 $\frac{1}{2}$  inches diameter, runs at a speed of nine turns per minute, gearing direct through a pair of beveled gears into the vertical drum shafts."

*Method Pursued in Determining Friction.*—Of the work done by the engine in operating a draw-bridge, part gives acceleration to the mass and produces velocity of rotation in the draw-span, the remainder being absorbed in friction. It was found impracticable to "indicate" the engines on account of the oscillating type, also on account of the variation of the velocity and the short time of runs, the engines being in operation less than two minutes. Therefore another method was pursued, as follows: The engines were run until the bridge attained a certain velocity; they were then thrown off and the bridge allowed to come to rest. The force producing this retardation must equal the force producing the work of friction. All runs being made during absolutely calm weather, no consideration of wind effect was necessary.

*Description of Apparatus for Determining Velocities.*—A continuous record of the velocity of the bridge was kept upon a strip of paper, which being drawn through by rolls moved by the main driving shaft of the bridge, moved proportionately to the velocity of rotation of the bridge. Upon this strip of paper a mark was made by a pencil point attached to the armature of an electro-magnet, the magnet being charged every two seconds by an electric contact made by a torsion pendulum; equal intervals of time were consequently marked upon the paper. The paper used was a heavy quality of manilla cut in strips  $3\frac{1}{2}$  inches wide and about 30 feet long. A number of records of swings could be taken upon these strips, as both sides were used. A strip was rolled up and placed on a spindle at the end of the guide box. This was simply a box closed upon three sides and open at the ends, through which the paper was fed. A block of wood, pressed upon by an adjustable spring, pressed the paper against the bottom of the box, producing the desired tension. From this box the paper passed to the marking apparatus, the paper passing through guides and over a marking plate as it passed under the pencil point. The paper was then led to the feed rolls. These consist of an arbor, accurately lined and firmly fastened, upon which turns a well-fitted sleeve, this sleeve being turned by a pulley fastened to it, which is driven by a belt from the main driving shaft. A loose roll, pressed down upon the sleeve by means of springs and being prevented from turning about the spindle, it, together with the sleeve, drew the paper, which was led along between them, at a rate proportional to the turning of the bridge.

A torsion pendulum was used, as the length of a simple gravity pendulum which would give the required interval of time, was too great to allow of its being swung on the bridge; and as no suitable clock could be procured, and a torsion pendulum would run quite accurately for the short time (three minutes) required for a swing. The pendulum was located upon the stone pier under the radial spider rods. This position was so chosen because the vibrations of the bridge were less and because the motion of the bridge could not influence the swinging of the pendulum in any manner. The pendulum was constructed of two well seasoned pieces of pine, fastened together in the shape of an inverted T and suspended by a piece of tempered steel wire. Upon the cross-piece and equidistant from the center were placed two equal weights, and by varying their distance from the center of oscillation the time of vibration might be adjusted. The weights were adjusted by timing the pendulum by a watch and counting the clicks of the magnet for six minutes; in that time the pendulum would not vary a tenth part of a second. The whole was incased in a box to protect it from draughts, etc., a glass being placed in the top so that the operation of the pendulum might be observed. The suspension wire passed through a close-fitting hole in a sheet of metal at the top of the box, and was fastened to a horizontal piece of metal, a movement of which, in a horizontal plane, would give a purely torsional movement to the wire. The electric contact was made by means of a wire leading from the torsion wire and a drop of mercury in an adjustable block of wood. The mercury was placed in the center of oscillation and the contact wire passed through it twice during each swing.

*Method of Calculating the Moment of Inertia.\**—The moment of inertia,  $I$ , of the draw was calculated by breaking the whole into parts as follows, the total  $I$  being the sum of these parts:

*First.*—Inertia of the truss, the weights of the drum, etc., being considered as part of the truss.

- (a.) About an axis traversing the center of gravity of the truss parallel to the axis of rotation. This was found by multiplying the weight of the truss, reduced to joint loads, by the squares of their distances from the axis of the truss.

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\* All the weights used in the calculation of the moment of inertia are the actual shipping weights of the members, as weighed at the shop.

- (b.) The total weight of the truss into the square of its distance from axis of rotation.

*Second.—Inertia of floor beams.*

- (a.) About an axis traversing the center of gravity parallel to the axis of rotation.
- (b.) The weight of each floor beam into the square of its distance from the axis of rotation.

*Third.—Inertia of stringers, rails, and guard-rails.*

- (a.) About an axis traversing the center of gravity parallel to the axis of rotation.
- (b.) The weight of these pieces into the square of their perpendicular distance from the axis of rotation.

*Fourth.—Inertia of timber, ties, etc.*

Considered as a flat rectangular strip of uniform weight per cubic foot about the axis of rotation.

CALCULATION OF THE MOMENT OF INERTIA. 1 a and 2 b.

Truss No.	Distance.	$\frac{2}{2}$ Distance.	TRUSS.		FLOOR BEAMS.	
			Weight.	Product.	Weight.	Product.
0.....	248.74	61 851.69	17 700	1 094 775 090	18 400	828 812 780
1.....	224.91	50 580.01	26 410	1 335 817 800	8 830	446 621 400
2.....	201.07	40 441.21	25 660	1 037 721 192	8 830	367 095 796
3.....	177.24	31 399.84	29 660	931 318 068	8 830	277 260 234
4.....	153.41	23 381.56	30 160	705 189 056	8 830	206 459 528
5.....	129.57	16 796.16	33 660	565 360 092	8 830	148 310 446
6.....	105.74	11 172.49	38 160	426 342 600	8 830	98 653 175
7.....	181.91	6 707.61	34 660	232 485 416	8 830	59 226 108
8.....	58.07	3 375.61	39 660	133 876 296	8 830	29 806 548
9.....	34.24	1 169.64	38 660	45 216 736	8 830	10 327 568
10.....	10.41	108.16	92 660	10 025 812	8 830	955 406
	Sum .....	.....	407 050	6 518 128 158	.....	2 463 530 989

This is for only half of one truss, and the first total product should therefore be multiplied by four for the total. It is also for half of the span for the floor beams, and this product should be multiplied by two for the total. The trusses are 14.16 feet from the axis of rotation.

2 a. For simplicity's sake consider each floor beam to consist of a rectangle  $27 \times 4$  feet  $\times 2$  inches; then since  $I = \frac{1}{3} W(b^2 + c^2)$ , in which  $W = 8 830$ ,  $2b = 27$  feet and  $2c = 2$  inches,  $I = \frac{1}{3} \times 8 830 \times 182 257 = 536 440$ . There are twenty-two floor beams.

3 a. Consider a rectangular strip  $500 \times 2.5$  feet  $\times 2$  inches; then since  $W = 90\ 533$ ,  $2b = 500$  feet,  $2c = 2$  inches,  $I = \frac{1}{3} \times 90\ 533 \times 62\ 501.563 = 1\ 886\ 170\ 000$ .

There are four stringers, etc., two distant 3 feet and two distant 9 feet from the axis of rotation.

4. For the floor beams  $W = 206\ 667$  and the dimensions are  $500 \times 18$  feet  $\times 9$  inches; hence  $I = \frac{1}{3} \times 206\ 667 \times 62\ 824 = 4\ 102\ 560\ 000$ .

#### SUM OF I.

1 a.	$6\ 518\ 128\ 158 \times 4 =$	26 072 512 632
1 b.	$407\ 050 \times (14.16)^2 \times 4 =$	326 454 100
2 b.	$2\ 463\ 530\ 989 \times 2 =$	4 927 061 978
2 a.	$536\ 440 \times 22 =$	11 801 680
3 a.	$1\ 886\ 100\ 000 \times 4 =$	7 554 400 000
3 b.	$90\ 533 \times 3^2 \times 2 =$	1 629 594
3 b.	$90\ 533 \times 9^2 \times 2 =$	14 666 364
4		4 102 560 000
		<hr/>
		43 011 086 348
		<hr/>

#### WEIGHTS.

Trusses, drum, etc., $4 \times 407\ 050$	1 628 200
Floor beams, $8\ 830 \times 20 + 13\ 400 \times 2$	203 400
Stringers, rails, etc., $90\ 533 \times 4$	362 132
Timber	206 667
	<hr/>
	2 400 399
	<hr/>

$$\rho^2 = \frac{I}{W} = \frac{43\ 011\ 086\ 348}{2\ 400\ 000} = 17\ 921.286$$

$$\rho = 133.87 \text{ feet.}$$

*Determination of the Acceleration.*—Having found  $\rho$  the radius of gyration, it was necessary to find the negative acceleration at  $\rho$  distance from the axis of rotation, in order to determine the force producing the negative acceleration, which is the frictional resistance. From the proportion of the paper rolls, etc., it was found that 1 inch of paper was equivalent to 3.1218 feet at  $\rho$ . The acceleration in inches was found from the formula  $f = \frac{2s}{t^2}$ , in which  $f$  = the acceleration,  $s$  the distance in inches of paper corresponding to that from the throwing off of the engine to the point of cessation of motion of the bridge, and  $t$  the number of seconds indicated on the strip between these points. This was multiplied by 3.1218 to give the acceleration at  $\rho$  in feet. This was done for each card, and the average acceleration taken. As the friction is assumed

to be constant, the acceleration is consequently assumed to be constant. The table shows, however, variations which apparently follow no law. These include slight variations in friction for different portions of the track, unobservable wind pressures, imperfections of apparatus and errors of observation. The mean of so large a number of observations must, however, give a value probably nearly correct.

TABLE OF ACCELERATIONS FROM CARDS REDUCED TO  $\rho$  DISTANCE FROM CENTER.

Card No.	Time = $t$ in seconds.	(Time) <sup>2</sup>	Length card in inches = $s$ .	Acceleration in feet. $f = \frac{2s}{t^2} \times 3.1218$
5.....	56	3 136	8 438	.01680
6.....	48	2 304	6 438	.01745
7.....	80	6 400	15 375	.01601
8.....	82	6 724	17 750	.01655
9.....	86	7 396	17 250	.01456
10.....	100	10 000	23 375	.01460
11.....	78	6 084	16 188	.01662
12.....	48	2 304	5 688	.01541
14.....	118	13 924	29 469	.01320
15.....	116	13 456	36 000	.01666
16.....	102	10 404	24 219	.01454
17.....	144	20 736	45 594	.01373
18.....	134	17 956	40 188	.01397
19.....	90	8 100	17 547	.01356
20.....	118	13 924	28 422	.01285
21.....	110	12 100	23 875	.01233
22.....	142	20 164	44 531	.01379
24.....	74	5 476	14 563	.01663
25.....	44	1 936	5 844	.01886
26.....	80	6 400	16 719	.01533
27.....	82	6 724	16 094	.01493
28.....	92	8 464	19 906	.01469
29.....	124	15 376	32 313	.01312
$\div 23 =$		Sum.....		.34419
$\div 23 =$		Average.....		.01496

#### Frictional Resistances at Center of Rollers—

Let  $R_1$  = radius to center of rollers = 16 feet. Let  $F$  = force at extremity of rollers = 16 feet. due to  $f$ .

$\rho$  = radius of gyration = 133.87 feet.  $f$  = acceleration.

$F_1$  = force at center of rollers  $W$  = weight of bridge.

due to  $f$ ,  $\phi$  = total co-efficient of friction.

$$\text{Then } F_1 = F \frac{\rho}{R_1} = Mf \frac{\rho}{R_1} = \frac{W}{g} f \frac{\rho}{R_1}$$

$$F_1 = \frac{2400000}{32.2} \times .015 \times \frac{133.87}{16} = 9354$$

$$\phi = \frac{F_1}{W} = \frac{9354}{2400000} = .00389$$

This total amount of friction is due to:

1. Collar friction of bearings on live ring.
2. Overhauling shafting and gearing.
3. Rolling friction.

*Collar Friction of Bearing on the Live Ring.*—The friction due to the pressing of the roll against the live ring cannot be determined experimentally. The theoretical assumption made, however, which cannot depart far from the truth, was that the outward horizontal pressure will be the component of the weight of the bridge, pressing vertically, due to the obliquity of the rolls. This would be absolutely true if the wheels and track were absolutely true and frictionless. Of course this cannot be realized in shop practice. If the apex of the conical rollers falls beyond the center of rotation, the roll will tend to move outward when the bridge turns, increasing the outward pressure. If it falls short of the center, conversely, it will tend to roll in, decreasing the pressure. These will tend to neutralize each other, and, together with the jarring and vibrating of the bridge and the rolling action of the rolls, may cause the full theoretical horizontal component. Assume that this is so, then let  $H$  = the horizontal component,  $W$  = weight of bridge,  $H = 2 W \tan \alpha$ , and we have the total outward strain  $H = 2400000 \times \frac{2 \times 10}{16 \times 12} = 250000$ .

For collar friction we have (see Rankine, A. M., page 251). Lever arm of friction =  $\frac{r^3 - r_1^3}{r^2 - r_1^2} = 1.7$  inches.

Assume  $\phi_2 = .07$  and let  $W_1$  = work done by collar friction of live ring bearing for 90 degrees swing, then

$$W_1 = 2 \pi \times \frac{1.7}{12} \times .07 \times 250000 \times \frac{\pi \times 12 \times 32}{4 \times \pi \times 20} = 74760 \text{ foot-pounds.}$$

*Work Lost by Gearing.*—From "Konstrukteur" — F. Reuleaux.

$$p_r = \pi \phi_3 \left( \frac{1}{Z} + \frac{1}{Z_1} \right) \frac{E}{2}.$$

$\phi_3$  = co-efficient of friction = .15.

$E$  = length of arc of contact =

$p_r$  = loss of work.

$1.6 \times$  pitch (Rankine, M. M.).

$Z$  and  $Z_1$  = number of teeth.

Then for bevel gears,  $E = 2.51$ , and

$$p_r = 3.14 \times .15 \left( \frac{1}{17} + \frac{1}{45} \right) \frac{2.51}{2} = .048.$$

For pinion and rack,  $E = 5.4$ , and

$$P_r = 3.14 \times .15 \left( \frac{1}{360} + \frac{1}{13} \right) \frac{5.4}{2} = .10.$$

*Work Done in Overhauling Horizontal Shaft.*—Weight of shaft = 2500 pounds.

For 90 degrees swing the shaft turns 18.33 times.

$W_2$  = the work done by the horizontal shaft for 90 degrees swing.

The distance the shaft moves through = 4.5 inches  $\times \pi \times 18.33$  = 259 inches. Assume  $\phi = .05$ , then

$$W_2 = .05 \times \frac{259}{12} \times 2500 = 2698 \text{ foot-pounds.}$$

*Friction in the Collar of the Vertical Shaft.*—The lever arm of the friction = 6.4 inches,  $\phi_2 = .07$ . Weight of shaft = 1 000.

$W'_2$  = work absorbed in friction for 90 degrees swing of the shaft turning 6.92 times.

$$W'_2 = 6.92 \times \frac{2 \pi \times 6.4}{12} \times 1000 \times .07 = 1622 \text{ foot-pounds.}$$

*Total Work Done by Overhauling Shafting, etc.*—

$$\frac{2698}{(1-.05)} + 1622 = 5053 \text{ foot-pounds for 90 degrees swing.}$$

*Relative Work Done by Friction.*—This work for 90 degrees swing of the bridge is:

	Foot-pounds.
For collar friction of the live ring bearings.....	74 760
" overhauling gearing, etc.....	5 053
	79 813

$$\begin{aligned} \text{The total work of friction} &= 9354 \times \frac{1}{4} \times \pi \times 32 \\ &= 234972 \text{ foot-pounds.} \\ &\quad \underline{79\ 813\ " " } \end{aligned}$$

$$\text{Work of rolling friction} = \underline{\quad 155\ 159\ " " }$$

The percentage of friction due to—

$$1. \text{ Collar friction of live ring bearing is .....} = \frac{74\ 760}{234\ 972} = 31.8 \text{ per cent.}$$

$$2. \text{ Overhauling, shafting, etc....} = \frac{5\ 053}{234\ 972} = 2.1 \text{ "}$$

$$3. \text{ Rolling friction .....} = \frac{155\ 159}{234\ 972} = 66.1 \text{ "}$$

*Co-efficient of Rolling Friction.*—

$\phi_1$  = Co-efficient of rolling friction.  $w$  = work.

$F_2$  = Force required to move  $W$ .  $S$  = space moved through in 90 degrees of swing.

$$\phi_1 = \frac{F_2}{W} \quad w = F_2 \times S; \text{ hence}$$

$$\phi_1 = \frac{w}{S \times W} = \frac{155\ 159}{\frac{1}{4} \pi \times 32 \times 2\ 400\ 000} = .00257.$$

*Another Method for Determination of  $\phi$ .*—The determination of the total friction by moving the bridge by the application of a force at the end, was accomplished in the following manner: From the end of the draw, a line was attached leading over a sheave at the end of a strut run out from the fixed span; at the end of this line weights were placed, until the bridge just moved at a uniform velocity. Hence if  $\phi$  = co-efficient of friction;  $F$  = weight applied = 530 pounds;  $F^l$  = weight if applied at the track circle =  $530 \times \frac{250}{16}$ ;  $W$  = total weight of bridge.

$$\phi = \frac{F^l}{W}; \quad \phi = \frac{\frac{250}{16} \times 530}{2\ 400\ 000} = .00345$$

Considering the crude manner in which this last experiment was performed, as the uniform velocity was judged only by eye, and as it considered only a small section of the track, the weight, 530 pounds, was the mean of a large number of runs, being quite a variable quantity; it agrees approximately with the first and more accurate method—11 per cent. difference, showing the first figure to be not far from the truth.

## PART II.

### METHOD FOR DETERMINING POWER TO OPERATE DRAW-BRIDGES.

It is proposed to discuss only that class of draw-bridges in which the whole weight of the structure is supported by a drum resting upon friction rolls, and the center pin only serves to prevent lateral motion. The surfaces of contact of the drum, rolls and supporting track are conical, and have a common vertex in the axis of the drum. The rolls are held in place relatively to the center by spider rods. These rods are fastened to a collar on the center-pin cap, and pass through the rolls. Outside the rolls, and on the rods, are collars or washers, against which the rolls bear. Anchored to the pier, outside the track, is a large gear. Two pinions on vertical shafts, diametrically opposite to each other, mesh with this gear for the purpose of swinging the bridge. The vertical shafts are connected through bevel wheels with a horizontal shaft driven through gearing by the engine.

*Resistances.*—The resistances to be overcome in swinging a draw are friction, inertia and wind pressure. The frictional resistances are rolling friction, sliding friction between the collars on the spider rods and the rolls, and friction of gearing. The resistance of inertia is the force required to accelerate the motion of the bridge. The resistance of wind pressure is indeterminate, and can only be approximated. If the wind blew perfectly steady, it would have no effect upon the swinging of the bridge, as the pressure on one side of the center would equalize that on the other, whatever the direction of the wind. The wind, however, never blows steadily, but in gusts, and locally, so that for a long span draw-bridge the velocity at one end seldom equals that at the other. When it is desired that the draw should be operated quickly, it is necessary that the engines should be capable of developing sufficient power to swing the draw, when the maximum and minimum velocities of the highest wind under which it is considered safe to operate the draw are affecting the opposite ends at the same time. The effect of wind was very noticeable while conducting the experiments on the Thames River bridge; as, in high winds, and with the throttle at a constant opening, irregular accelerations and retardations could be noticed in the increase and decrease in the revolutions of the engines.

In determining the resistances it will be most convenient to determine their equivalents at the pitch circle of the large gear or rack, as it is called, since it is, at this point that the force to overcome them is applied.

*Rolling Friction.*

Let  $R_1$  = radius to center of track. Let  $F_r$  = force at rack required

$R$  = radius of rack.

to overcome rolling

$W$  = weight of bridge.

friction.

$\phi_1$  = co-efficient of rolling friction.

$$\text{Then } F_r = \phi_1 W \frac{R_1}{R}$$

*Collar Friction.*—For the friction of washers and collars at the end of the spider rods.

Let  $r_1$  = interior radius of collar. Let  $W$ ,  $R$ , and  $R_1$  be as above.

$r_2$  = exterior " " "  $F_c$  = force at rack to overcome

$r$  = radius of rolls.

collar friction.

$\phi_2$  = co-efficient of friction.

Then the force with which the rolls are pressed against the col-

$$\text{lars} = W 2 \frac{r}{R_1}$$

From Rankine, A. M., we have the lever arm of the friction or the radius of the circle at which the total friction may be considered to act :

$$= \frac{2}{3} \times \frac{r_2^3 - r_1^3}{r_2^2 - r_1^2}.$$

The ratio of the force at the rack to the force at the track required to overcome the collar friction  $= \frac{R_1}{R}$ .

$$\text{Then } F_c = \frac{2}{3} \times \frac{r_2^3 - r_1^3}{r_2^2 - r_1^2} \times \frac{1}{r} \times \frac{R_1}{R} \times \phi_2 W \times 2 \frac{r}{R_1}$$

$$= \frac{4}{3} \frac{r_2^3 - r_1^2}{r_2^2 - r_1^2} \frac{\phi_2}{R} W.$$

*Friction of Gearing.*—For this we have from Konstructeur—F. Reuleaux :

$$p_r = \pi \phi_3 \left( \frac{1}{Z} - \frac{1}{Z_1} \right) ea.$$

Where  $p_r$  = percentage of work lost in transmission through gearing.

$\phi_3$  = co-efficient of friction.  $a$  = a constant depending upon  $z$  and  $z_1$  = number of teeth of gears. the nature of the teeth.

$e$  = arc of contact.

Let  $p_1 p_2 p_3 \dots \dots \dots$  be the percentages of work lost in the transmission through the several pairs of gears,  $w$  = the work done at the rack and  $w_1$  = the work done by the engine, which is the sum of the work done at the rack and the work lost in transmission through the gearing.

$$\text{Then } w_1 = \frac{W}{(1 - p_1)(1 - p_2)(1 - p_3)}$$

In subsequent calculations, it will be convenient to consider the gearing frictionless, and increase the actual resistances at the rack in such manner as to make the work of the engine the same as when the actual resistances at the rack and the friction of gearing are considered. Thus, letting

$F_1$  = the sum of the forces to be overcome at the rack;

$F$  = the force at the rack required to overcome  $F_1$ , and assuming the gearing to be frictionless;

$$\text{Then } F = F_1 \frac{1}{(1-p_1)(1-p_2)(1-p_3) \dots \dots \dots}$$

In this equation the factor  $\frac{1}{(1-p_1)(1-p_2)(1-p_3)}$  is a constant for a given draw-bridge. Calling it  $a$ , we have  $F = a F_1$ .

*Inertia.*—To determine the force required to overcome the inertia of the draw, it is necessary to determine its moment of inertia in reference to the axis of rotation. This, by a well known principle of mechanics, will be the sum of the moments of inertia of each part in reference to an axis through its center of gravity, parallel to the axis of rotation of the draw, plus the sum of the mass or weight of each into the square of its distance from the axis of rotation. A practically accurate method is that used in the experiments on the Thames River Draw-bridge. Sufficient accuracy may be obtained by considering the draw to be a rectangular parallelopipedon of uniform density, of the same weight as the draw, and whose length and breadth are the same as those of the draw, and whose depth, or dimension parallel to the axis of rotation is any convenient quantity.

To find the mass which, placed at the radius of the rack, would produce the same moment of inertia:

Let  $R$  = radius of rack;

$M$  = mass at  $R$ ;

$I$  = moment of inertia of draw; then

$$M = \frac{I}{R^2}$$

And letting  $F_1$  = force required to accelerate the draw, or overcome its inertia,

$f$  = the acceleration at the rack;

then

$$F_1 = Mf.$$

*Wind.*—To find effect of wind, let  $p$  = the pressure on any member due to the difference of maximum and minimum velocity of the highest wind under which the draw is to be operated, and suppose the wind to blow perpendicular to the length of draw. Then let  $f$  = the distance of the center of pressure of any member from the vertical plane through the center of rotation and parallel to the direction of the wind; and let  $F_w$  = a force which placed at the rack would produce a moment equal to the moment of the wind pressure. Then, since  $R$  = the radius of the rack, we have

$$F_w = \sum_o^n f p$$

in which  $n$  = the number of members exposed to wind pressure on one end of the draw. With sufficient accuracy we may consider the total wind

pressure as acting at a distance  $l$  from the center = one-quarter the length of the draw, when, letting  $P$  = the total wind pressure, we have

$$F_w = \frac{P l}{R}.$$

*Design of Engine.*—Since the force required to swing the draw is the sum of the forces required to overcome the various resistances, we have from preceding formula, letting  $F$  = this force,  $F = a(F_r + F_c + F_w)$ .

In the second factor of the right member  $F_r$ ,  $F_c$  and  $F_w$  are constant for a given bridge, so calling their sum  $c$ , we have  $F = a(c + F_1) = a(c + Mf)$ .

*Acceleration.*—To find the acceleration on the assumption that the draw is uniformly accelerated for the first half of the swing, and uniformly retarded for the last half, let  $t$  equal the time in seconds in which it is required to swing the bridge, then

$$f = \frac{2S}{\left(\frac{t}{2}\right)^2},$$

in which  $S$  = one-eighth the circumference of the rack = one-eighth ( $2\pi R$ ).

$$\therefore f = \frac{2\pi R}{t^2} \text{ and } F = a(c + M \frac{2\pi R}{t^2}).$$

If it is desired to operate the draw by accelerating it until it attains a given velocity which is to be maintained until the brakes are put on and the bridge brought to rest:

Let  $T$  = time required for swing; Then  $L = (l + l_1)$  = space in which

$L = \frac{1}{2}$  circumference of rack; the draw is to have

$v$  = velocity at circumference uniform velocity;

of rack;  $t$  = time of acceleration;

$l$  = space on circumference of  $t_1$  = time of uniform velocity;

rack in which draw is to  $t_2$  = time of retardation;

be accelerated; then  $T = t + t_1 + t_2$

$l_1$  = space on the circumference of the rack in which the draw is to be retarded;

$$\text{but } t = \frac{2l}{v}, t_1 = \frac{L - (l + l_1)}{v}, t_2 = \frac{2l_1}{v}; \therefore T = \frac{L + l + l_1}{v}.$$

In this equation  $l$  and  $l_1$  must be assumed, when either  $v$  or  $T$  may be assumed and the other found. We have

$$f = \frac{v_2}{2l}; \therefore F = a \left( c + M \frac{v^2}{2l} \right).$$

*Dimensions of Cylinders and Multiplication of Force by Gearing.*—In the designing of engines for draw-bridges the size of engine may be limited on account of the small space for machinery. In this case the multiplication of force by gearing must be determined.

Let  $v$  = greatest linear velocity at rack;

$m$  = ratio of length to diameter of cylinder;

$n$  = greatest number of revolutions per minute of engine;

$F$  = force at rack, as in previous formulas;

$\frac{l}{2}$  = length of crank;

$N$  = number of revolutions of the engine to one of the draw;

$R$  = radius of rack;

$p$  = steam pressure.

The work per second at the engine must equal  $Fv$ , and

$$Fv \cdot 2pl \cdot \frac{\pi m^2 ln^2}{4 \cdot 60}; \text{ but } \frac{n}{60} = \frac{v}{2\pi R} N; \therefore N = \frac{4RF}{p l^3 m^2}.$$

In this equation  $p$  and  $m$  must be assumed, when either  $l$  or  $N$  may be assumed and the other found.

If the speed of the engine is to be restricted by piston speed instead of revolutions per minute, let  $S$  = maximum piston speed per minute;

$$\text{then } 2p \pi \frac{m^2 l^2}{4} \frac{S}{60} = Fv, \text{ but } \frac{S}{60} = \frac{2ln}{60} \text{ and } \frac{n}{60} = \frac{vN}{2\pi R};$$

$$\therefore N = \frac{2FR}{m^2 l^3 p}. \text{ The maximum horse-power will be } HP = \frac{Fv}{550}.$$

*Constants.*—The co-efficient of rolling friction  $\phi_1$ , as determined in the experiments on the Thames River Draw-bridge, was .00257, or in round numbers .003. The co-efficient  $\Phi_2$  for collar friction may be given the ordinary values for sliding friction. In the formula for friction of gearing,  $\Phi_3 = 0.20$  and  $a = \frac{1}{2}$  for cycloidal and three-quarters for involute teeth. In the formula for effect of wind pressure,  $p$  may be taken as the pressure due to a wind velocity of 30 miles per hour, which it is believed will be ample.

A good thumb rule for power to operate draw-bridges would be to assume a co-efficient of rolling friction large enough to allow for all resistances. In case of careful construction, a co-efficient of .01 would be ample. For cheap or hasty work .015 should be allowed. To find the horse-power required let  $W$  = weight of bridge,  $v$  = maximum velocity per second at circumference of rack, then  $HP = \frac{.01 \text{ or } .015 (Wv)}{550}$ .

## DISCUSSION.

THEODORE COOPER, M. Am. Soc. C. E.—The Second Avenue Draw-bridge in New York City is operated by hydraulic power, operating through wire ropes running over sheaves and attached to the axis of the turn-table wheels. Diameter of track circle, 26 feet. Points of attachment of hauling ropes are on a circle of 27 feet 4 inches diameter. There are four rams, 9 inches diameter of plunger and 6 feet long. Two rams open and two close the draw. The weight upon the track circle is as follows:

	Pounds.
Shop weights of iron and machinery.....	638 263
Rails, spikes and handrail.....	36 750
Ties, footwalks, etc.....	204 987
Total weight .....	880 000

Soon after the completion of the bridge, experiments were made to determine the frictional resistance. It was found that after the bridge was started from the position of rest, 100 pounds on the rams would keep it in motion, 100 pounds on the two rams = 12.723 pounds. This by means of the sheaves was reduced to 3 344 pounds acting at the circumference of the wheel, or  $\frac{3\ 344}{880\ 000} = .0038$ , or 3.8 pounds per 1 000, as the force necessary at the circumference of the wheels to overcome all the resistances, including rolling friction, friction of the collars, rams and sheaves. This is almost identical with the amount found by the authors for the Thames River Draw-bridge.

As the estimated force necessary to overcome the inertia for a speed sufficient to open the draw in one minute was 5 400 pounds applied at the center of the track, we needed for the friction and inertia 8 744 pounds, which required 260 pounds on the rams. The rams, accumulators, etc., were designed for a pressure of 400 pounds, and are usually worked to about 350 pounds, in order to have a surplus for wind resistances. The Watertown tests for 1881 give the following results of experiments on the rolling friction of 1-inch steel rollers between cast iron plates under pressures per lineal inch of rollers varying from 1 250 to 3 750 pounds.

Co-efficient of friction—minimum, .00256; maximum, .01212.

Two-inch steel roller, tested in same manner and same pressures.

Co-efficient of friction—minimum, .00125; maximum, .00675.

Four-inch cast roller with pressures from 1 250 to 6 250.

Co-efficient of friction—minimum, .00175; maximum, .0040.

Same under 10 000 pounds pressure per lineal inch.

Co-efficient of friction—minimum, .0045; maximum, .0050.

A French experiment with a 4-inch iron roller loaded with 200 to 370 pounds per lineal inch gave—

Co-efficient of friction—minimum, .008; maximum, .012.

Same sprinkled with fine sand—

Co-efficient of friction—minimum, .026; maximum, .048.

By WILLIAM H. BURR, M. Am. Soc. C. E.—Although draw-bridge construction on a scale requiring a reasonably accurate determination of the power to be used in operating the turning and locking machinery is happily comparatively infrequent, yet the results of the investigation made by Messrs. Boller and Schumacher are of great practical value. Essentially nothing of the kind has heretofore been done, and while the details of their apparatus are not shown, it may be confidently assumed that nothing was omitted or neglected in connection with it that could conduce to greater accuracy or completeness than that attained. All features of the computations required in the interpretation of the experimental data satisfy the most exacting conditions, and engineers may congratulate themselves on at last possessing something in connection with swing-bridge resistances on which quantitative deductions entitled to a high degree of confidence can be based.

In determining the steam engine capacity for a given draw or swing bridge, it will not be necessary to divide the resistances, except to give a separate expression to that of the wind, nor to compute the moment of inertia of the structure in the detailed manner given in the paper, although the degree of refinement exhibited is strictly appropriate to the purposes of the investigation. By the aid of the total co-efficient of friction, or what may perhaps be more accurately termed the co-efficient of internal resistance of the structure, some very simple expressions for the power required to operate a draw can be easily written. For this purpose it will be assumed that in opening the draw the acceleration of motion (*i. e.*, the velocity gained each second of time) is constant until the highest permissible velocity is reached; after which the maximum velocity remains constant until retardation begins, which latter is taken to be constant until motion ceases; and these assumptions express the actual conditions as nearly as, or nearer than, any other that can be made. A swing span square crossing requires the structure to be turned through 90 degrees in being opened, and the same in being closed. It is not practicable to assign the exact limits to the periods of acceleration, uniform motion and retardation which comprise the total time necessary for the complete 90 degrees of motion, but it is not far wrong to assume that the power of the engine is exerted over one-quarter of the 90 degrees in producing the uniform acceleration which culminates in the maximum velocity, and that retardation takes place over the last fifth of the same 90 degrees, leaving fifty-five hundredths of the path for uniform motion. The wind considerably affects those proportions, but I have frequently observed approximately those just given.

Following the authors of the paper as closely as possible, the notation will be:

$W$  = total weight of the structure in pounds, including turn-table, rollers, etc.

$R_1$  = radius of drum in feet =  $\frac{1}{2} D$ .

$f$  = acceleration in feet per second of extreme end of the structure.

$r$  = retardation in feet per second of extreme end of structure.

$v$  = maximum velocity in feet per second of extreme end of structure.

$l$  = total length of structure in feet.

$w$  = transverse width of bridge between truss centers.

$\alpha$  = maximum angular velocity =  $\frac{2v}{l}$ .

$t$  = total time in seconds required to open bridge.

$g$  = 32.2 approximate constant for gravity.

Under the conditions assumed the extremity of the bridge would pass over the distance  $\frac{\pi l}{4} (0.55 + 2 \times 0.25 + 2 \times 0.2) = 0.3625 \pi l$  in the time  $t$ , if the velocity were uniform.

$$\therefore v = \frac{0.3625 \pi l}{t}; \text{ or, } t = \frac{0.3625 \pi l}{v} \quad (1)$$

$$\text{or, } \alpha = \frac{0.725 \pi}{t}; \text{ and } t = \frac{0.725 \pi}{\alpha} \quad (2)$$

According to what has just preceded, the acceleration  $f$  will exist during the time—

$$\frac{2 \times 0.25}{0.55 + 2 \times 0.25 + 2 \times 0.2} t = \frac{50}{145} t = 0.345 t.$$

$$\text{Hence : } f = \frac{v}{0.345 t} = \frac{1.05 \pi l}{t^2} \quad (3)$$

$$\text{And the angular acceleration } \alpha = \frac{2.1 \pi}{t^2} \quad (3a)$$

In the same manner the retardation will be—

$$r = v \div \frac{40}{145} t = \frac{v}{0.276 t} \quad (4)$$

In determining the moment of inertia of the structure about the vertical axis of revolution, it will be sufficient to consider the trusses, lateral bracing, floor system and track as a homogeneous prism with length  $l$  and width  $w$ . This portion of the weight is, for all practical purposes,  $\frac{1}{6} W$ . Its moment of inertia will then be—

$$I^1 = \frac{\frac{1}{6} W (w^2 + l^2)}{12 g} = \frac{5 W (w^2 + l^2)}{72 g} \quad (4)$$

The moment of inertia of the drum, rollers, etc., will be determined by considering the weights of those parts concentrated at the distance  $R_1$  from the axis. Hence that moment will be—

$$I' = \frac{1}{6} \frac{W R_1^2}{g} = \frac{W R_1^2}{6g} \dots \dots \dots \quad (5)$$

Hence the total moment of inertia is—

$$I = I' + I'' = \frac{W}{72\alpha} (5w^2 + 5l^2 + 12R_1^2) \dots \dots \dots (6)$$

The velocity of the extremity of the draw at the beginning of the last unit of time (*i. e.*, the last second) of the period of acceleration is—

$$v - f = v \left(1 - \frac{1}{0.345 t}\right).$$

Hence the path passed over by that point during that unit of time is—

$$v - f + \frac{1}{2}f = v \left(1 - \frac{1}{0.69t}\right).$$

The corresponding path under the rollers is—

$$\frac{2 R_1 v}{l} \left(1 - \frac{1}{0.69 t}\right) = \frac{0.725 \pi R_1}{t} \left(1 - \frac{1}{0.69 t}\right).$$

Hence the work performed during the same unit of time in overcoming friction is—

$$F = \frac{0.725 \phi W \pi R_l}{t} \left(1 - \frac{1}{0.69 t}\right) \dots \dots \dots (7)$$

If the unbalanced wind pressure of  $p$  pounds per lineal foot is taken to act on the entire length of one arm and constantly normal to the longitudinal axis of the structure, the center of that total pressure will be at the middle point of the arm, and the work performed in overcoming it during the same unit of time to which equation (7) applies, will be—

Finally the work expended in accelerating the motion of the structure is, by equations (3a) and (6)—

$$E = \frac{I \alpha^2}{2} = \frac{0.0306 \pi^2 W}{g t 4} (5 w^2 + 5 l^2 + 12 R_1^2) \dots \dots \dots (9)$$

The maximum amount of work, therefore, to be performed in a unit of time (*i. e.*, one second) is—

$$F + A + E = \frac{\pi}{t} \left( 1 - \frac{1}{0.69 t} \right) (0.725 \phi W R_1 + 0.0906 p l^2) + \frac{0.0306 \pi^2 W}{g t^4} (5 w^2 + 5 l^2 + 12 R_1^2) \dots \dots \dots (10)$$

The horse-power required by the engine will then be—

After the greatest velocity  $v$  is acquired, this work of the engine will immediately drop to that required to overcome the friction and wind resistance. The brake power required in retardation over and above that part of the latter caused by the friction, can readily be expressed by the same general method as the preceding, but it has no special practical value.

The investigations of Messrs. Boller and Schumacher show that for the Thames River bridge, the total co-efficient of friction was 0.004, very closely. Hence other data from the same structure are—

$$l = 500 \text{ feet} \quad w = 28\frac{1}{3} \text{ feet} \quad W = 2\,400\,400 \text{ pounds.}$$

$$t = 180 \text{ seconds} \quad R_1 = 16 \text{ feet} \quad p = 100 \text{ pounds.}$$

By equation (6) the moment of inertia becomes—

By inserting the remaining data, as just given, in equation (10)—

$$\text{Hence, } H.P. = \frac{41162}{550} = 75 \text{ horse-power} \dots \dots \dots (14)$$

It is interesting and important to observe that only about 5 per cent. of the total power is required to develop the acceleration and overcome the internal resistance; the remaining 71 horse-power is entirely applied to overcome the wind pressure. This latter has been taken at 10 pounds per square foot on an assumed area of 10 square feet per lineal foot of structure. These computations demonstrate the importance of the part played by wind pressure in the operation of swing bridges. The co-efficient of friction found by Messrs. Boller and Schumacher undoubtedly applies only to the unusually excellent grade of workmanship which characterized the Thames River construction. For all ordinary cases it will probably be best to make  $\phi = 0.01$ , for rim bearing tables. But even with this increased co-efficient the horse-power required for friction and acceleration in the present case would be but  $\frac{4820 + 27}{550} = 9.$

A comparatively small amount of power is, therefore, required to operate a swing bridge if there is no wind or if the wind pressures on the two arms are balanced.

If  $P$  is the total steam pressure on the piston of the engine,  $s$  the stroke and  $n$  the number of revolutions for the same unit of time as that to which  $\alpha$  applies, and if steam follows the piston the full stroke, as the greatest wind pressure may require, the total work of the engine per unit of time will be  $2 n s P$ . The work of the wind pressure and fric-

tion during the same time will be  $\left(\frac{p}{2}l + \frac{l}{2R_1} + \phi W\right) \alpha R_1$ . Hence motion will begin to decrease and the engine will eventually be stalled if  $P < \left(\frac{p}{4} \frac{l^2}{R_1} + \phi W\right) \frac{\alpha R_1}{2ns}$ .

Precisely the same general method as the preceding may be applied for the determination of power required for swing spans under any division of acceleration, uniform motion and retardation periods.

LEFFERT L. BUCK, M. Am. Soc. C. E.—Although this paper is in itself a valuable contribution, its value will be greatly increased should its discussion bring out the results which others have obtained from ex-

periments in a like direction, as it should do. The literature of the draw-bridge would then be increased in a direction in which it is at present greatly lacking. It might be well to request others to send in to the Society such data upon this subject as they may possess. It appears to me that the greatest value of the present paper lies in the fact that it gives the total resistance to motion of the draw, with the exception of that part due to inertia, which, in this case, was the force used in measuring the remaining resistances. The aggregate of the resistances is what we want most.

The authors of the paper are undoubtedly correct in saying that it is difficult to estimate the resistance of friction of the rollers against the live ring. But this is not necessary, unless for the purpose of trying to devise some substitute for the live ring. It is doubtful if the weight of the draw is directly an element of the pressure against the ring. The angle of the curve is so far within the limiting angle of friction that no weight could press the wheels out. It is imperfect workmanship that causes them to press out against the ring. If the workmanship was theoretically perfect and the wheels were placed either outside or within their true position, their tendency would be to roll into it. The coefficient derived from the experiment of attaching a rope to the end of one arm of the draw, passing the other end over a sheave and suspending a weight to it, even though the apparatus was crude, probably does not exceed that arrived at by the other experiment by more than the resistances of the sheave and of inertia, which last was in this case a resistance, especially if we consider that the starting friction is greater than the moving friction.

F. W. SKINNER, M. Am. Soc. C. E.—I have enjoyed and appreciated the paper of Messrs. Boller and Schumacher, which is valuable for the careful and laborious analysis and computations in a subject very little, if any, discussed or considered in a manner available to engineers. The theoretical character of the paper and its exclusively mathematical value make it difficult to discuss off-hand. Perhaps, too fine drawn for the practical working conditions of such cases as the one considered, but may be none the less valuable for the analysis of the problem and to indicate methods and steps of its general solution.

I agree with the other speakers that it needs to be reinforced with parallel investigations, and trust that some of our practical bridge engineers will present further data of similar character. To pass from the grave to the ridiculous, without comparison with the important subject of the paper, I may mention a humorous example of resistances in swing bridges that is suggested by recollections of patent highway swing bridges that I once knew to be, and probably still are, somewhat numerously manufactured by an Ohio bridge company. They were light country bridges, of perhaps 35 to 45 feet span, over canals. The swing was center bearing, and had a tall pivot with double cylin-

drical or conical walls, between which double spiral springs (of, perhaps, 1-inch square steel) were coiled in opposite directions, so as to oppose the swinging of the bridge in either direction from its axis. On each side of the bridge, along its lower chord, a longitudinal wooden buffer timber was placed, against which passing canal boats struck violently, forcing the bridge open, and scraping their way through, after which the spring vibrated the bridge until it finally stopped in a closed position.

W. H. BREITHAUPT, M. Am. Soc. C. E.—This paper is a valuable contribution to draw-bridge literature in that it gives concise data on something that has mostly been a matter of rough approximation only, and in this goes into an almost wholly new field. I cannot add much directly in the line of the subject of the paper, but might give an illustration of the obstruction by varying wind pressure on opposite ends to the turning of a long draw span, as spoken of in the paper. In the Fort Madison bridge, over the Mississippi River, C. S. F. & C. Ry., there is a 400-foot draw. This bridge carries a railway and a wagon road on either side on outer brackets. As it was desirable to cut off the view of trains from the roadways, 6 feet 6 inches of open slatwork screens were interposed between the roadways and the railway track. In figuring on the turning machinery, the weight of the draw span, which is a very heavy one, and regular wind surfaces, were allowed for. It was found, however, that the screens referred to, adding 3 feet or more per running foot to the effective wind surface as they do, made it extremely difficult to operate the draw during a wind, so that the turning machinery had to be reinforced.

In heavy draw-bridges it is of great importance to have the distribution of dead load over the turn-table uniform; to have the drum and the cross girders which transmit the weight to it, or to it and the center, stiff and rigid; to have the wheel track perfectly level and on an unyielding support, and to have all machine work accurately done. I see no good reason why the turn-table should always be preferably rim bearing only. When a large part of the weight can feasibly be transferred to the center it is well to do so, as it can from there more easily be distributed with uniformity to the masonry, and there is the further advantage of shorter traverse of moving parts there, and relatively greater accuracy obtainable in the working surfaces, resulting in greater ease of turning.

#### CONCLUDING REMARKS BY THE AUTHORS.

The reason the total resistance to motion of the draw was not given, as suggested by Mr. Buck, was because this resistance is made up of the constant resistance friction and the variable resistance necessary to overcome the inertia. The determination of the former was the main object of the paper; the latter, varying with each velocity, may be found readily for any given velocity.

The method employed by Mr. Cooper upon the Second avenue bridge is practically the same as the last method used by us as a check upon the first method. It is gratifying to see the co-efficient determined by Mr. Cooper agrees so very nearly with that determined for the Thames River bridge.

In figuring the moment of inertia Mr. Burr assumes  $\frac{1}{6} W$  in the trusses, flooring, etc., and  $\frac{1}{3} W$  in drum. In the Thames River bridge the weights are nearly—

Trusses, flooring, etc.....	2 185 000
Drum, etc.....	215 000
<hr/>	
	2 400 000

Which is nearer  $\frac{1}{2} W$  and  $\frac{1}{2} W$ .

Substituting these values then—

$$I = \frac{1}{3} 2 185 000 (250^2 + 14^2) + 215 000 \times 16^2$$

$$I = 45 717 170 000,$$

which is nearly (within 5 per cent. of) the figure previously found by the longer and more accurate method, *i. e.*:

$$I = 43 011 086 348.$$

The computation for wind pressure as assumed by Mr. Burr is probably excessive. By Smeaton's formula, given by Trautwine, 10 pounds per square foot corresponds to a wind of nearly 50 miles per hour. This wind is considered to blow entirely upon one end of the draw, which would require a severe hurricane blowing, and in that case the draw would not be required to be opened; 30 miles an hour would be ample to allow, which would correspond to 4.5 (say 5) pounds per square foot. The area of 10 square feet to 1 foot of draw is a large estimate for the Thames River bridge. But allowing this, then

$$\begin{aligned} F &= 1 920 \quad H. P. = \frac{22 470}{550} = 40 \\ A &= 19 609 \\ E &= \frac{941}{22 470} \quad \frac{19 609}{22 470} = 87 \text{ per cent.} \end{aligned}$$

Forty horse-power is somewhat nearer the capacity of the engines than 75, as determined by Mr. Burr. There are two oscillating engines, 10-inch diameter, 7-inch stroke, steam full stroke, say at 40 pounds pressure, and at 150 turns per minute.

$$\text{The horse-power} = 4 \frac{40 \times 7 \times 78 \times 150}{12 \times 33 000} = 33.$$

The analysis of Mr. Burr, it is very clear, shows from how many different points this question may be viewed. It would be well if the profession could have more discussions like this one. We desire to express our thanks for the kind manner in which this paper has been received and discussed, and hope that now the field has been opened additional data upon this subject may be developed.

Our object in dividing the resistances in Part I of the paper was to arrive at the rolling friction. In Part II we considered division of the resistances necessary, because the frictional resistances are constants, determined by experiment; wind is practically arbitrary, and inertia varies with the time required to operate the draw. We have given a very simple formula at the end of Part II, which is:

$$\text{Maximum horse-power} = \frac{.01 \text{ or } .015 (W v)}{550}.$$

The constants are more than sufficiently large to swing the bridge, but in case of a necessity for an allowance for wind and rapid operation of draw they should be doubled or trebled. This is made evident by the fact that in swinging the Thames River draw the steam pressure under ordinary circumstances was about 40 pounds, which, with the dimensions and revolutions of the engines, gave about 30 horse-power; and substitution in the above formula, considering maximum velocity at end of draw 3 feet per second, gave about 10 horse-power.

The information as to the usual periods of acceleration, uniform velocity and retardation in practice are very valuable, and will give practical value to the general formula under the head "Acceleration" in Part II. Mr. Burr's analysis differs from our own chiefly in the fact that he uses angular acceleration where we use linear, and takes as his unit of moment of momentum the unit of mass where we use the unit of weight. The suggestion to consider the span as a homogeneous prism was anticipated in Part II, under the heading "Inertia." The value  $\phi = .01$ , suggested by Mr. Burr, was recommended by us at the end of Part II.

The necessity for considering the friction of the rolls against the live ring is evinced by the fact that the spider rods which bear the collars against which the rolls press, and which support the live ring, occasionally break; and it is impossible, with the most accurate machinery with which the rolls and track can be made, to secure theoretical accuracy, under which condition only there will be no lateral tendency of the rolls. Even if machinery could be devised to procure theoretical accuracy at first, unequal wear and settlement of pier and track would soon destroy the theoretical accuracy.

# AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

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## TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

518.

(Vol. XXXV.—December, 1891.)

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### RED ROCK CANTILEVER BRIDGE.

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#### FOUNDATIONS.

By S. M. ROWE, M. Am. Soc. C. E.

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#### WITH DISCUSSION.

As a formal report has already been made to the Atlantic and Pacific Railroad Company, through its General Manager, D. B. Robinson, the purpose of this paper will be to outline the history of the construction of the bridge; the causes that led to its construction, and to give all facts that may be of interest to the civil engineer and the scientist. The original line of the Atlantic and Pacific Railroad, when constructed in 1883, was laid in such a way as to skirt and traverse the valley of the Colorado River for a distance of nearly 8 miles, finally crossing that river about  $2\frac{1}{2}$  miles south of what is now Needles Station, on the California side. This point of crossing is 18 miles south of Fort Mohave, near which is fixed the southernmost point of the State of Nevada. Ten miles south of the original bridge, the river valley narrows into what is known as Mohave or Needles Cañon; so named from a small but rugged

range of volcanic mountains on the Arizona side. At the head of this cañon is the site of the Red Rock Bridge.

The Colorado River rises in southern Wyoming, draining western Colorado, eastern Utah, a small portion of southern Nevada and eastern California, and about three-quarters of the Territory of Arizona. The drainage area is about 230 000 square miles, most of which is mountainous, consequently its water is always turbid, and the quantity of silt held in suspension and carried forward at the same velocity as the water, is very considerable, amounting, as determined by precipitation of numerous samples, to 1.56 grains per cubic inch of water. The silt when thoroughly dry is found to weigh 59.95 pounds to the cubic foot.

While most of the distance through which the river flows is across a desert, almost rainless (except on rare occasions in which storms of great violence occur), its tributaries rise mainly on the west slope of the Rocky Mountains, where heavy rains are frequent and where the snowfall is sometimes quite heavy. Large accumulations of snow suddenly set free by a sudden thaw or by heavy rain, owing to the great declivity of the land, will cause violent floods and correspondingly severe attrition both on banks and river bed, by which earth, sand, gravel and boulders will be carried forward.

To form some idea of the amount of silt carried forward, take the fine silt above mentioned, much of which is so fine that it requires hours or even days to precipitate it. In the high water of 1884, taking the record as given by Chief Engineer W. A. Drake, who estimated the maximum velocity at 8 miles per hour (which seems from more recent observation to be within bounds), and an approximate area of river at 48 500 square feet; then assuming 5 miles per hour as the mean velocity, giving 384 000 cubic feet of water per second, the 1.56 grains per cubic inch would give 7 900 000 cubic yards of earth, or 6 400 000 tons at 1 620 pounds per cubic yard for each twenty-four hours of that flood. Then taking 50 000 cubic feet per second (about the mean for the year) as the mean amount of water passing, we would have about 160 000 000 cubic yards per day. Add to this the movement of the heavier sands and gravel, which is constantly going forward (though at less velocity), both by the movement of sand bars in the river bed and from cutting away of the banks, and it will be seen that this river is a powerful excavator as well as disintegrator. The Grand Cañon stands as a notable monument of this. In the case in hand, the line, as built, was subject to the latter mode of attack, and from the

great depth to which the cutting extended (20 to 40 feet), it was found impracticable to protect the road-bed.

The valley of the Colorado River, for a long distance above the Needles Cañon, is not more than 12 to 20 feet above the ordinary water level, and has been traversed to its bounds by the river channel, and no doubt will continue to be so. Indeed, quite radical changes are taking place yearly. The greatest flood occurs in June, and is usually due to melting snow in the Rocky Mountains; but rises of 3 to 5 feet may occur at almost any time in the season, and are liable to come with little or no warning, the source being so far distant that they do not correspond to the local weather indications in the least. The declivity of the river from Fort Mohave to the Needles Cañon, and for a mile or two into the cañon, is about  $1\frac{1}{10}$  feet per mile, though probably greater below that point, as the river is quite narrow where it cuts through the Needles Range.

The volume of the river varies from a minimum of 5 000 cubic feet per second to probably 500 000 in extreme high water (elevation 469.6 to 500.5). While the danger from this was not unforeseen by the locating engineer, Mr. Lewis Kingman, its gravity was not fully appreciated. The line finally adopted was surveyed by him in 1880, and the cost was estimated at \$666 220, while the line then built was estimated at \$586 554, making a difference of \$79 666 in favor of the latter. The line *via* Red Rock, then designated as the lower crossing "C," was identical with the point of crossing selected by the Kansas Pacific engineers in 1867, and also by the Atlantic and Pacific engineers in 1871.

The experience of the six years since the completion of the original line was such as to again force the attention of the management to the necessity of some effective remedy, and a joint letter from William B. Strong, President of the Atchison, Topeka and Santa Fé Railroad Company, and E. F. Winslow, President of the St. Louis and San Francisco Railroad Company, was issued January 30th, 1888, directing A. A. Robinson and James Dun, Chief Engineers of the respective companies, to examine with regard to what improvement was necessary to put the line in proper condition.

Leaving out many minor matters relative to the improvement of bridges, buildings, track, equipment, etc., we quote from the report made by these two chief engineers representing the joint ownership of the road, so much as relates to the question in hand.

## COLORADO RIVER CROSSING.

"One of the most serious questions which we have to report upon is the crossing of the Colorado River. We find upon examination, that the present line is unsafe; that a portion of it is below recent high water mark, and all of it is subject to the encroachment of the Colorado River, so that it will be impossible upon the present line to give you any reliable figures as to the present or future cost of maintaining the present line and bridge, except to say that we know it will be very great.

"The estimate for the change of line and the maintenance of the present crossing, as shown on Exhibit 'C,' is based on the highest water known, but it is possible that the data are insufficient to establish the true high water mark; in which case there would be a constant element of danger from this source, which is entirely eliminated by adopting the Red Rock Crossing line. During the last six months the river has encroached upon the line about midway between Powell and the present bridge, threatening the line for more than a mile. This action of the river is undoubtedly only the beginning of the working over of the bottom into a new channel, as is customary with rivers of this class.

"While the expenditure of forty or fifty thousand dollars at this time might protect the road during the year 1888, it is more than likely that the same expense would have to be incurred during 1889, and indefinitely thereafter, with the more than probable danger of having our line cut in two entirely, and traffic suspended several weeks or months. Under this condition of affairs we see only one remedy, that is, to place the crossing of the Colorado River upon a safe and permanent basis.

"Two plans suggest themselves; one of which is to build the line directly across the valley, instead of following up the middle of the valley as by the present line, and building your track upon this bank from the present bridge to the bluff on the east side of the valley; thence along the foot of the bluffs and connecting with the present line near Powell. This plan will enable us to use the present bridge for some time to come, by lengthening it 500 or 600 feet. The road could be built upon this new location for about \$160 000. This, however, would leave us with the river to fight, to maintain the channel under the present bridge. This expense of maintaining the channel from year to year will continue, and would undoubtedly amount to several thousand dollars each year. In addition to this, the character of the Colorado River is such that it is almost impossible to maintain a channel at any fixed point. The river is a navigable one, and the bridge is built upon a low grade line. The draw span is not serviceable, because the main channel of the river passes under the bridge several hundred feet from the draw. As a result of this you are paying a steamboat company now

plying the river, \$500 per month, because of the obstruction of the river by our bridge, and are unable to let boats pass. This is a dangerous situation, and leaves the gate open for any enterprising individual to buy an old rotten tub, and dispose of it to advantage by blackmailing the railroad company. The amount which we are now paying this steamboat company, namely, \$6 000 a year, represents the interest on \$100 000 at 6 per cent. The chances are, as we have intimated, the day is not far distant when more will be demanded.

"These considerations have led us to the conclusion that we can only say that the best course to pursue is to reconstruct the line for about 13 miles, crossing the river on a high grade line at the Red Rock crossing, about 9 miles below the Needles Station. This would put the line forever upon a safe basis and stop the further expenditures for protecting your road-bed and bridges against the river and for damages to navigation, and, in our judgment, is the only safe and wise course to pursue.

"We could not, of course, construct this bridge in time for the freshet of 1888; consequently, the expense of maintaining your present line during this season would have to be incurred. What this will be no person can tell until the freshet comes. The work which has already been done, seems to us to have been judiciously done. The money has been well expended, and is all that can be done until the high water season develops what further expenditures are necessary.

"Exhibit 'C' herewith gives you an estimate of the cost of reconstructing this 13 miles of road and the permanent bridge, and also the cost of rebuilding and putting in a permanent structure on the present bridge site and changing the line from Powell as above indicated.

"We would recommend that in doing this work, the grading be contracted for completion by the 1st of August; that the contract for the bridge be made for delivery on the 1st day of November, and that the track be laid to the bridge by the middle of September; that the work upon the masonry for receiving the superstructure be completed on November 1st, and that the bridge be erected and ready for traffic by January 1st, 1889. It is possible that the expenditure might be delayed, and the interest on the investment saved for three months more, but considering the difficulties liable to be encountered, the distance of the undertaking from market, and the extreme heat\* which prevails in that climate, we think the plan should be laid as above indicated, to make sure of having the structure completed in ample time for the freshets of 1889. Thirteen miles of new steel will be required on this work.

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\* Heat; maximum atmospheric 124 degrees Fahr.; of sand, 135 to 145 degrees, and metal 165 degrees; minimum atmospheric, + 26 degrees Fahr.—*Rowe*.

NOTE.—During the summer of 1888 the river encroached both above and below the bridge, and with 3 feet more water, the line would have been lost and the river channel would have crossed the line about 3 miles east of the bridge. Luckily the summer flood was less than usual; notwithstanding this, the track was saved only by an expense of over \$500 per day for over a month.—*Rowe*.

"We are sorry, indeed, to have to recommend to you the expenditure of so large a sum of money. We are certain, however, that a great mistake has been made in locating the line upon its present road-bed, and so we believe that the only course now open for this company is to rectify this mistake; that to continue to maintain the road-bed in the present position will only result in loss and disaster, which will be many times greater than the work of placing the line upon the proper location. There is no question whatever in our minds, but that this is the proper course to pursue."

The river bed had been examined systematically by Louis Trainor, Engineer of the Southern Pacific Railroad Company, in 1881, at which time that company was engaged in building eastward from Mojave. The maximum depth at midstream to reach rock was made about 30 feet, and it was assumed that by using a 400-foot span at midstream, the depth of foundation would not exceed 24 feet.

**COMMENCEMENT OF SURVEY.**—On July 2d, 1888, the writer received orders to make a survey and estimate of the proposed line crossing at Red Rock, and on the 4th walked over a portion of the line, making 15 miles, and on return to Needles Station at 6 p.m., found the thermometer standing at 122 degrees Fahr. This is simply mentioned so that the conditions may be to some degree appreciated by those unacquainted with the climate of that locality. The surveys were prosecuted vigorously, and only the river soundings remained to be done; these only awaiting arrival of appliances ordered and the subsidence of the summer flood, now about passed. On September 14th, in response to urgent request from the management, the following report was transmitted through the General Superintendent, A. A. Gaddis:

PRELIMINARY REPORT OF SURVEY OF RED ROCK LINE.

"ALBUQUERQUE, N. M., September 13th, 1888.

A. A. GADDIS,

*General Superintendent.*

I submit, in accordance with your directions, contained in your letter of July 2d, a report of survey, estimate and map (Plate CXI) of the proposed change of line, and of crossing of the Colorado River at Red Rock, as recommended by Messrs. Robinson and Bond, in their report submitted to Presidents William B. Strong and Edward F. Winslow, in February of the present year.

**The Line.**—Commencing at Station 14 571 + 41, a point 3050 feet west of mile-post 562, or mile 562.6 of the main line of the Atlantic and Pacific Railroad; we run immediately across the Sacramento wash, and supporting on the side hill we reach the river immediately opposite

to what is known as Red Rock. Crossing the river, we are obliged to curve at the earliest practicable point, to avoid precipitous hills, using 9 degrees curvature, being the lightest practicable. Thence we run up the edge of the river valley, keeping at all points above extreme high water marks, and avoiding possible attack from the river, and, at the same time, keeping the grade line sufficiently above the beds of the washes from the hills west to insure the passage of water under the track. We connect again with the present main track just west of the present bridge, at Station 312 + 84.5, at a distance by present line from the point of starting of 54 156.9 feet, having run by the new line 70 184.6 feet, making an increase in distance of 3.036 miles. A preliminary profile is herewith submitted, showing curvature and grades, hastily laid, from which to deduce approximate quantities.

Before proceeding with work the line should be slightly shifted in several places, and the grades revised. The approximate quantities given I feel quite sure will be somewhat reduced in construction, but will serve as a basis of estimate. The desire to save any unnecessary expense and not to delay the transmission of this report impels us to submit the matter in a less perfect state than is desirable. Most, if not all the items are intended to be above, rather than below what the actual cost should be.

*Grading.*—In estimating the cost of grading, I assume that 20 cents per cubic yard for waste and borrow; 26 cents for excavation hauled; 64 cents for loose, and \$1 for solid rock excavation, and 1½ cents per cubic yard hauled 100 feet, will be a fair estimate for this work. The quantity of solid and loose rock will be small.\* In view of the approach of the favorable season, these figures can probably be shaded somewhat. Water is convenient and supplies in easy reach. A detailed statement of quantities is appended.

*Pile and Trestle Bridges.*—Owing to the numerous washes making into the river, particularly on the west side of the river, the amount of pile bridging will be large. I think there is no place (case) where piles cannot be driven. It will require about eighty pile bridges, varying from 30 to 345 feet in length, with a mean height of 10 feet in the clear. These I would estimate at \$6 per linear foot.

*Iron Bridge at Red Rock.*—It was intended to make full and careful soundings of the river bed at the line of crossing, but as it required at the present stage of water, expensive equipment and considerable time, it was deemed best to rely for preliminary estimate on the soundings made by Louis Trainor in 1881. These soundings were made under very favorable circumstances, the river being, at that time, nearly 6 feet lower than at present.

As the rock in question, on which it is proposed to build, is a volcanic "breccia," forming a dyke across the river at this particular place, the

\* Most of the material to be moved is sand and loamy clay, and in no case marshy.

depth is liable to great variation, even in the length of the pier (about 60 feet); and I would advise careful examination by sounding before the line is fully established across the river. A slight shifting may be very advantageous and save much expense. The river is gradually falling, and where a great volume of rapid flowing water was passing two months ago, to the depth of 20 feet, is now a mass of sand and mud with not over 8 feet of water at the deepest point.

*Plan of Bridge.*—I submit plans for piers for the channel span, which may prove too light. I will examine them later with reference to this. They will be exposed to the force of water and drift only, as there is no ice in the Colorado River; but at extreme high water this will be considerable, as the water has at some time—years ago—attained a depth of from 50 to 60 feet, and a very high velocity.

*Stone for Masonry.*—I think the Chino stone should be used and the masonry should be first-class, laid with the best cement. The English Portland can be bought, I understand, at San Francisco, for \$3.50 per barrel; adding freight, 50 cents, would deliver it at the Needles for \$4, being about as cheap as domestic cement could be laid down at that point, and it will, I think, be most economical and give the best results for the cost.

After October 1st, on and until the May following, the river will be low, and I think there will be no serious obstacles to prevent the sinking of cribs for the two channel piers and the erection of the falsework for raising the bridge. The work cannot be delayed after the 1st of October or possibly a month later. From May 1st until about the present time it is impracticable to undertake such a work.

*Relating to Erection of the Bridge.*—I would advise the building of the line from the Needles south to the bridge site, as material can be delivered on west side of the river more conveniently.

In relation to the 396-foot channel span there might be a saving by using a shorter span, but the manner in which the channel shifts would seem to indicate it as being safer than a shorter span. From my observation of the river during high water, I think the span, as located, will always cover the main channel.

*Superstructure.*—By courtesy of the Keystone Bridge Company I am able to give their maximum figures for the erection of the superstructure of the bridge and the iron trestle approaches (Plate CXII). The quantities given for the masonry are the contents of the piers as shown by plan. The price assumed at \$12 per cubic yard seems high, but I doubt whether Chino stone can be put in any cheaper, as it is as hard and nearly as heavy as granite.

A sketch for a caisson is submitted (Plate CXIII), showing a section across a pier and manner of weighting it down (Plate CXIV). My estimate for 886 cubic yards of beton is for this purpose and to fill the inner space when the pier is completed. As you will perceive, this will increase the

base of the pier at least 3 feet on all sides, and obviate largely the necessity of riprap. I think the weighting in this manner is the best, as it strengthens, and at the same time renders the wall of the caisson watertight.

**GRADING 13½ MILES:**

Bankment.....	261 346.8	cubic yards, at	20 cts., \$52 269 36
Excavation hauled.....	72 968.6	" " "	26 " 18 971 84
Loose rock excavation .....	2 332.7	" " "	64 " 1 492 93
Solid rock excavation.....	6 529.4	" " "	\$1 6 529 40
Excavation hauled 100 feet... .....	594 778.9	" " "	1½ " 8 021 69
Total.....			\$87 285 22
Pile bridges (78) aggregating 7 185 linear feet, at \$6 per linear foot .....			43 110 00

**MASONRY:**

Bridge masonry.....	4 212.73	cubic yards, at \$12,	\$50 552 76
Pedestal masonry.....	115.22	" " "	15, 1 728 30
Cement, E. P.....	2 165.	barrels, " 4,	8 660 00

Total.....			60 941 06
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**FOUNDATIONS:**

Cost of two caissons.....			\$5 826 00
Labor and machinery sinking foundation.....			10 600 00
Labor on other piers and abutments.....			3 800 00
False work for bridge .....			9 500 00

Total.....			28 626 00
Beton for filling cribs, 886 cubic yards, at \$8.....			7 088 00

**SUPERSTRUCTURE:**

One span, 396 feet, at \$160 per linear foot.....			\$63 360 00
Two spans, 195 feet 8½ inches, at \$89 per linear foot.....			34 818 68
281 linear feet iron trestle, at \$33.60.....			9 441 60

Total.....			107 620 18
Engineering and exigencies .....			15 000 00
Freight on iron, Albuquerque to Needles, 975 tons at \$5.76.....			5 616 00
Freight on Chino stone, 9 100 tons at \$1.42.....			12 922 00

Then add as per report of Messrs. Robinson and Bond—

3 miles new steel rails, 50 pounds.....			\$16 060 00
9 600 new ties, at 72c.....			6 840 00
Laying and surfacing 13 miles track.....			10 000 00

Total.....			32 900 00
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Grand total.....			\$401 308 46
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(Signed) SAM'L. M. ROWE, Resident Engineer.

Following the transmission of the foregoing report, the location of the line was completed except in the immediate vicinity of the crossing of the river, and requisitions were made for machinery, boat and cable necessary to do the sounding; but in view of the considerable cost of the appliances and in the absence of any knowledge whether the work was likely to proceed, the local management hesitated to incur the expense,

hence this part of the work was delayed until orders were received to let the work. The only thing then practicable was to presume on the correctness of the Trainor soundings, and so let the work, providing for such changes in the bridge structure as circumstances would make necessary. Facing the necessity of prompt action, to be able to finish the bridge before next season's high water (it now being the 20th of November), it was decided to employ Waddell and Jenkins, professional men at that kind of work, and possessing the necessary appliances, to do the sounding. Mr. Jenkins of this firm was promptly on the ground and proceeded to vigorously prosecute the boring. At the first test boring at station 193 + 75, where it had been planned to locate the west channel pier, it was found that instead of 22 feet to bed rock, the drill passed with but little interruption, to a depth of 64 feet. Meantime, the grading was let and well under way and the superstructure and substructure, both let provisionally; the former to the Phoenix Bridge Company, the latter to Sooysmith & Co. On making this discovery, all proceedings under these contracts were stopped and a thorough examination of the river bed was arranged for with the contractors for the soundings. Orders were issued to make a sounding every 50 feet so as to develop a complete section of the river on the line fixed upon. These soundings were completed January 1st, 1889.

When it became apparent that the Trainor soundings were entirely unreliable, and while the new conditions were being developed by the progress of the soundings, various combinations, using various lengths and numbers of spans, were carefully compared and the cost estimated approximately. While the plan seemingly best adapted to all the conditions of the case—that of two through spans of 405 feet each—did not largely exceed the cost of the various combinations of shorter spans, yet two considerations following were regarded as formidable in the opinion of the writer hereof, who at that time filled a subordinate position with relation to this work. In the first place, spans of such unusual length would be heavy and require a much larger amount of metal, and necessitate the maximum amount of masonry, as all the piers would have to be built to the grade line nearly 80 feet above low water, and to proportionately increased dimensions. This for the reason that the depth from grade to high water was insufficient to allow either span to be used as a deck span. In the second place, taking the known history of the river and the depths to bed rock at the point where the pier at midstream

would be, it would be subjected to unusual stress from the great depth of water flowing at the excessive speed of 12 to 14 miles per hour and to a depth of nearly 100 feet. There is good reason to believe that the scour at that point has reached almost to the surface of the rock and that this probably occurred in the nowise extreme flood of 1884. The records of the Trainor sounding of 1881 certainly indicated that a stratum of boulders similar to that found in sinking the caisson at Station 189 + 25, did at that time extend entirely across the whole bed of the stream. By reference to Plate CXVI, it will be seen that the Trainor soundings and those of Jenkins conform very nearly, from the east bank to a point nearly one-third of the distance across the river (Station 190 + 50). The Jenkins soundings, however, found nothing of the kind beyond that point, giving plausibility to the conjecture that the flood of 1884 had removed all beyond the point mentioned. There is nothing above the line of the Jenkins soundings, except sand and gravel and a very few small boulders. Hence, while these objections were not deemed insuperable, yet it was considered best to survey the whole field before deciding. At the time that the chief engineers of the two companies visited the bridge site in February, 1888, the writer being present, suggested that, in view of the formidable character of the stream and the uncertainty as to foundation for piers in the river, the cantilever might be adopted. It was answered, however, that the cantilever was an expensive type of bridge, and if a foundation could be found at moderate depth, the truss would be cheaper.

When the Jenkins soundings had progressed so as to not only demonstrate the utter unreliability of those formerly made, but also to indicate that the foundation for at least one pier toward the east shore could be had at a very moderate depth, the question of the cantilever immediately recurred, and singularly enough, Mr. Waddell, agent of the Phoenix Bridge Company, came in response to a telegram, with a hasty but quite well digested plan for such a bridge (Plate CXV). The span lengths were fixed and orders telegraphed to Mr. Jenkins to thoroughly sound a pier site at Station 189 + 25. While this was being done, the plan of the superstructure was made and checked over simultaneously at Albuquerque and at Phoenixville, by aid of the telegraph, and on December 29th, 1889, a proposition was received from the Phoenix Bridge Company of a pound price as had been provided in the provisional contract, just four days after the matter was taken up.

Numerous soundings were made at and about the proposed site, and by the order of the writer, one center boring was pushed down so as to determine, if possible, the character of the rock. This was done and the depth of the boring into the rock below the surface reported at 5 feet. The rock increased in hardness to such an extent as to prevent further progress with the drill. The depth below low water did not exceed 14 feet in the borings on the outskirt of the site, or a mean depth of 12 feet for the foundation.

The plans for the substructure were prepared immediately and work was at once commenced by Sooysmith & Co., under an amended contract. The specifications were prepared by Consulting Engineer Prof. S. W. Robinson, of the Ohio State University, with the assistance of Prof. J. A. L. Waddell, on the part of the Phoenix Company, and by him accepted on the part of that company. The presumption being that it would be very difficult and expensive, if not impossible, to maintain the old line through the coming summer of 1889, everything was done with a purpose to complete the bridge by May, or, at furthest, June, of that year.

In this connection it may be said that the adoption of the cantilever was justified, even if more costly than the two spans of through trusses (which it was not, there being nearly \$60 000 in its favor), the bed of the stream being such that no prudent builder would take the risk of maintaining falsework during the season of high water, into which the work must, from the lateness of the time of commencement and the dimensions of the work, probably extend. Although, for causes hereafter stated, this danger was escaped in 1889, yet in the following year the wisdom of the plan was fully vindicated, as will be shown hereinafter. Then again, when the constantly shifting channel with reference to navigation, rendered it uncertain where the channel would be during the season of low water, one of the two plans became necessary to avoid the heavy demurrage by boat owners; that is, either the two through spans or the cantilever; and the former is still open to complaint during high water as a dangerous obstruction in the rapid river. Indeed, complaint was made to the Secretary of War in April, 1890, about the time the bridge was being completed, that the present structure would obstruct the navigation of the river; and Major W. H. H. Benyaurd, of the United States Engineers, was detailed to investigate. While the report has not yet been received, Major Benyaurd expressed his conviction to be,

"that a plan better adapted to provide for unobstructed navigation could hardly be devised." To still further show the wisdom of the course taken, it may be here noted that the June rise of 1890, did carry away the falsework used for erection of the east anchor arm, although very heavy and strongly built; happily, however, after the erection had progressed beyond the point of danger therefrom.

In January, 1889, a contract was made with Sooysmith & Co., whose pneumatic plant was arriving at the site, to build the masonry and the concrete anchor piers, and to sink one caisson at Station 189 + 25. For the masonry and concrete work the prices were not changed, as, while the amount of masonry as contemplated in the original contract was very much reduced, on the other hand, the amount of concrete work was considerably increased. The caisson,  $28 \times 57\frac{1}{2}$  feet, was built in place by the aid of the protection of an artificial breakwater, and sinking commenced. At the same time the contract for the cantilever and the viaduct approach was closed with the Phoenix Bridge Company, and arrangements made to prosecute the manufacture of the metal with the utmost speed.

The cantilever, as planned, was proportioned to conform to the grounds, the position and character of which in a great measure governed the economic location of the piers. Dividing the structure into two anchor arms of 165 feet each, and making the cantilever arms of the same length and the suspended span of even 330 feet, allowed the east anchor pier to be placed at low water level on solid rock at the east water edge; the west main pier, the same at the west water edge; the west anchor pier at the proper distance back and on a rocky ledge; and brought the east main pier on what was then deemed the best and most westerly location practicable without largely increased expense. It was generally conceded that the excessive length of 330 feet for the suspended span was not exactly the most economic; this was especially so claimed by Professor William H. Burr, Engineer of Construction for the Phoenix Bridge Company. After consideration, it was decided, for reasons above given, to adhere, however, to this arrangement, in which action Professor Burr readily concurred.

The following is given from the computations by Consulting Engineer Professor S. W. Robinson, showing that the objection to the span length as being uneconomical, had but little weight as compared to the cost of sinking to a foundation at greater depth both in and at each bank of the river.

"For the case that the center suspended truss is 330 feet or 275 feet, respectively; otherwise the same, except to be equally well proportioned for carrying the loads; we find that with the center span 275 feet, the whole bridge will weigh less than if it be 330 feet by 18.29 tons of metal, or there will be a less cost of \$2 194.80 at 6 cents per pound. The principal figures run about thus:

	330 feet.	275 feet.
Weight center trusses without track or floor —tons.....	271.42	180.81
Difference—tons.....	90.62	
Weight cantilever arms, not including track or floor or details common to both—tons,	402.04	447.00
Difference—tons.....	44.96	
Maximum moment at one main pier—foot tons.....	591 175 82 498.8	
Per cent. of excess in anchor arms for 275 feet center.....		5.72
For center span of 330 or 275 feet :		
Weight of supporting members of anchor arms—tons.....	478.5	505.87
Difference—tons .....	27.37	———
Total advantage for 275 feet center—tons...		90.62
Total disadvantage—tons.....	44.96	
Difference in favor of 275 feet center = (90.62 — 27.37 = 18.29) 27.37 .....	27.37	———
	72.33	90.62."

Then again the question was raised as to the proper distance between the trusses. While at the time of making the design the same conclusion had been arrived at independently both at Abuquerque and Phoenixville, viz.: to fix this at 25 feet, center to center; and work had reached such a state of progress as to render it very expensive if not impracticable, to change the design; yet in view of the great interest in the work represented by the party raising the question, it was determined to submit the matter to some undoubted authority. This was done with no purpose on the part of the writer (at this time Chief Engineer) to change the design, but with the hope of settling the question to the satisfaction of all concerned, as to the correctness of the conclusions arrived at by the designers, originally. Indeed, with the large amount of work already done on the superstructure, and the substructure also, as well as the question of time of completion of the bridge

and the danger threatening the traffic over the line, by which the loss, direct and indirect, would amount monthly to a very large sum, the question of change could not be entertained. The question was, however, settled to the general satisfaction of all, after full consideration. It is not claimed that the design is the very best possible, but taking into consideration the haste with which it had to be prepared, and the many points to be considered, the immense amount of computation and the amount of draughting and office work, it will do credit to all engaged. The simple, yet effective manner in which the suspended span is connected to the two cantilever arms, the nicety with which all expansion and contraction is provided for, the amplitude offered for lateral and transverse bracing, and the simplicity and effectiveness of the self-sustaining arrangements during erection, are some of the points that could be cited.

The following from William H. Burr, which is pertinent to this question, may be given here with propriety, as, while the Professor is connected in a business way with the Phoenix Bridge Company, still his standing as an authority on this subject will hardly be disputed.

"Abundant lateral stability is secured in an ordinary uncontinuous truss, if the width between centers of truss is taken at  $\frac{1}{8}$  the span. In fact, engineers frequently make the extreme distance between truss centers for long spans  $\frac{1}{6}$  the span length; but I think  $\frac{1}{8}$  is a better limit, as giving greater lateral stability. Now, by a calculation, which is needless to give here, it can be shown that if the suspended span of a cantilever opening is  $\frac{1}{4}$  of that opening, then a distance between the centers of a cantilever truss of  $\frac{1}{87}$  of the total opening will give essentially the same stability (lateral) as that possessed by a simple uncontinuous truss of the same length as the total cantilever opening with its truss separated by a distance equal to  $\frac{1}{8}$  the span. But  $\frac{1}{87} \times 660$  equal 25.7 feet, and our suspended span is one-half of the total opening, thus permitting the trusses to be a little nearer together, *i. e.*, 25 feet, instead of 25.7. Again, the chord of the cantilever and anchor arms are about four feet inside measure, making the total width of structure 29 feet, and materially enhancing its stability. For all these reasons, I have no hesitancy in saying that the Red Rock cantilever trusses are amply far apart."

The Professor might have added the further fact of the peculiar form of the bridge giving less depth of truss than usual at center instead of greater, as is the case with ordinary truss. Wind stress being the agent feared, it is proper here to say that, while the design was being made, communication was opened with Lieutenant W. A. Glass-

ford, then Superintendent of the United States Signal Service for the Pacific slope, who reported after examining the records of that department that—

"There is no record of wind so violent as to be characterized as a tornado, except in two cases, and in those the indications were that they were rather of the character of a sand gale."

At the bridge site, however, storms of this character are quite frequent, attaining a velocity of nearly 60 miles per hour, and frequently so strong as to stop work on the erection of the bridge. One occurring on the 27th of February, 1890, at which time the west half of the bridge was approaching completion, with the immense traveler on its long river arm, went very far to assure all observing the effect of the wind, that the bridge was stable. For the safety of the traveler it was lashed to the bridge, but the latter showed but a slight tremor. It had been proposed to stay the bridge with guy-lines to anchor cribs in the river, but it was found unnecessary to do so. Observation shows that the Arizona or Southern California wind storms, though severe, move with a steady flow, with scarcely a lull for hours at a time, and do not produce those sudden gusts that are found so destructive farther east. Indeed, what is known as the tornado or hurricane, is scarcely known west of the Continental Divide, or for several hundred miles eastward. While the usual provisions were made for wind stress, still there are less grounds for anxiety at that point than farther east.

Owing to the greater variation of temperature, unusual care had to be taken to provide for expansion and contraction of the metal. The lowest atmospheric temperature at that point is 26 degrees Fahr., and the highest, 125 degrees, making a range of about 100 degrees, causing a total difference in the length (660 feet) of the span between main piers of about  $5\frac{1}{2}$  inches (using co-efficient .00000663 per degree Fahr.). The bridge resting entirely on the two main piers, with the suspended span supported from the cantilever arm by four large eye-bars at each end, and with the lower chord telescoping after erection wedges were removed, this expansion is provided for, half at each end. At no time under this range of temperature can any stress be exerted on the main piers. The expansion on the anchor arms is provided for by a slight rocking motion of the long anchor bars which turn on the anchor pin at the bottom of the anchor pier. The possible danger from a cold weld, should it occur on these piers, is provided for by slightly increasing the size of

these eye-bars, beyond that necessary to bear the direct stress of the bridge.

While iron lying in or upon the sand, will absorb heat by exposure to the direct rays of the sun to 165 degrees Fahr., still in the bridge structure, it will not go over 135 to 140 degrees Fahr. This will not cut any figure, as there is an excess of some (over) 4 inches in the slotted chord, fully meeting such excess. Sand with the thermometer at 120 degrees in the shade, will absorb heat to 140 to 145 degrees, while water in the river rarely exceeds 60 degrees.

Resuming the history of the work: By the 16th day of March, the work having been steadily and uninterruptedly pushed, the grading had been completed to the bridge site at both banks of the river; the Chino Quarry had been opened and most of the stone cut and conveyed to the site; the west anchor and west main pier completed and the caisson built, and the sinking of the same had progressed to the depth of 8½ feet. The metal for the anchorage had been manufactured and received, and a large portion of the metal for the bridge had been rolled and was in process of manufacture; in short, there was every assurance that the bridge would be completed in June of 1889, when an unexpected difficulty was encountered.

The caisson, at the depth then reached, uncovered a portion of the supposed bed rock, which was found to consist of a compact boulder bed. Further investigation showed that there were portions of it that could be penetrated by the drill, showing gravel and sand underneath. Work on the caisson was immediately suspended and every appliance in reach was used to determine to what depth this extended. Several holes were drilled to a depth of 64 feet or thereabouts, below low water, all indicating a thickness of 3 to 4 feet in this bed of boulders, with nearly 20 feet of what seemed to be mainly sand and gravel, and with boulders frequently encountered, but of too unstable a character to depend upon for the foundation of the pier. Below this to 64 feet was found quite compact, yet of the same general character. After repeated trials with the drills (2-inch) then used, nothing beyond the 64 feet could be penetrated; even with 3-inch casing with tools of the same size, but a foot or two more could be made. Thus it became evident that a much stronger caisson, sunk to a much greater depth, would be necessary, involving an additional expenditure of at least \$100 000. The question of change of design could hardly be entertained for a moment, yet, in view of such a

large addition to the cost, it was deemed best to at once refer the matter to the management at New York and Boston.

The facts in relation to the soundings, on which the design was founded, will be given more at length hereafter, not to fix blame on any party connected with the work, but to serve as a lesson for the future. If any mistake was made beyond what might be due to the haste in which the work was done and to the unusual circumstances of the case or to the conditions there existing, the writer is not aware of it. Heavier and more perfect tools and appliances in the first place, would have gone very far to avoid the failure. These were afterwards used and the true bed rock was reached after penetrating 20 feet of large, smooth and very hard boulders, a result not often accomplished.

Meantime, work on the caisson and on the metal of the superstructure was immediately suspended, and by request of Presidents E. F. Winslow and William B. Strong, on behalf of the St. Louis and San Francisco, and Atchison, Topeka and Santa Fé Railroads, joint owners, it was arranged to have the matter investigated by A. A. Robinson, Second Vice-President, and James Dun, Chief Engineer. These gentlemen proceeded at once to examine the condition of the matter, reporting (a copy of which report is here inserted):

"ALBUQUERQUE, N. M., March 24th, 1889.

E. F. WINSLOW, President,

*St. Louis and San Francisco Railroad Company.*

WILLIAM B. STRONG, President,

*Atchison, Topeka and Santa Fé Railroad Company.*

"GENTLEMEN.—We have made a careful examination of the site of the proposed bridge across the Colorado River at the Needles, and of all the data in the office of the Chief Engineer of the Atlantic and Pacific Railroad Company, going over the estimate of cost of the various plans that have been proposed; with the result that we, without hesitation, agreed that the best and most economical course to pursue is to make no change in the plan of crossing, except to provide a larger and stronger caisson on which to found the east pier, and sink it to the depth of 70 feet, with a possibility of having to go to a maximum depth of 80 feet below low water. We are both of the opinion that it would not be safe to sink the caisson to a less depth.

"The soundings that have been made this season show that the boulders found above this depth are in nests of very limited dimensions and that

NOTE.—Confirmatory of this deep scour, it may be mentioned that about June 1st, 1890, shortly after the new bridge was opened to traffic, one pier of the old bridge went out, throwing two spans into the river. When it is considered that the pier in question had a pile foundation to a depth of 55 to 58 feet below water, it will be seen that the possibility of scour to great depth is fully demonstrated.—*Rowe.*

they are of comparatively small size, varying from 6 to 24 inches in diameter; neither do these collections of boulders appear to be stable. We have examined the records of soundings made at this same place (point) for the Southern Pacific Company by Mr. Trainor in 1881. He was engaged for a month in making the soundings and has left full records of the same, and evidently intended to make a thorough search for the bed rock and thought he had reached it. We can only account for the wide discrepancy between the soundings made by him and those recently made, on the supposition that the bed of boulders which Mr. Trainor found and reported to be bed rock, was scoured out by the exceptional flood of 1884, and that fine sand and gravel replaced them. The comparatively small size of the boulders found in the caisson of the east pier shows conclusively that they are shifted about from one point to another at the caprice of the current, and that the fine sand and gravel in which they are imbedded, affords no reliable foundation.

"We have made as close and reliable estimates as the information at hand justifies of the cost of the three plans, which seemed best to meet the new condition of affairs.

"*First.*—A cantilever span of 830 feet, the piers of which would rest on the rock, which is exposed on both sides of the river.

"*Second.*—Two spans of 415 feet each, resting on a pier midstream (and piers on rock on each shore).

"*Third.*—To adhere to the present plan of a cantilever span of 660 feet, with the exception of sinking a larger and stronger caisson 30 x 60 feet outside dimensions, to a depth of not less than 70 feet below low water.

"The comparative cost of the three plans resulted in favor of plan No. 3, aside from damage claimed by the bridge company owing to the change of plan of the structure. This damage, the bridge company estimate, would be about \$50 000, which renders the consideration of any other plan than No. 3 inadmissible. It is almost certain that the high water will occur by the 15th of May, and there is not sufficient time to procure the additional material and sink the caisson in the short interval remaining. We therefore recommend that the contract on the anchorage piers be allowed to proceed, as the stone is all quarried and nearly all cut, while the force and plant to do this part of the work are now on the ground and could not be brought back without an increase in expense. An extension of the time for completing the superstructure should be made until March 1st, 1890.

"The additional material for the caisson should be delivered in September, 1889, so that it could be framed in that month and the work of sinking be started by October 1st, 1889. We find that an agreement has been reached with Sooysmith & Co., for adjusting the value of the work performed by them to date, on the caisson which it is necessary to abandon. The sum agreed upon is \$24 000, which we believe to be fair

and just. The grading for the new line is completed with the exception of one rock cut at the west approach of the bridge. About two-thirds of the pile bridging is completed, the track being carried over on temporary cribs on the remainder. The track is laid and surfaced for the entire distance, except at the site of the bridge and through the cut above mentioned. The masonry for the west pier is completed and the foundation for the anchorage piers ready for the concrete. Nearly all the stone is quarried and cut for the masonry. All the work which has been done is first-class in every respect, particularly so as regards the masonry.

"The following estimate of cost of the bridge and road when completed is submitted, which we believe will be found to be in excess of actual results, viz.:

Engineering, etc.....	\$25 000
Grading .....	84 000
Pile bridging.....	48 000
Steel rails.....	12 100
Cross-ties.....	13 100
Track fastenings.....	6 500
Track laying .....	12 000
Masonry and foundation of west pier completed.....	14 700
Loss on abandoned caisson, east pier.....	24 000
Cost of new caisson, east pier, and sinking same.....	108 000
Cost of east pier.....	15 600
Cost of anchorage piers.....	33 600
Cost of superstructure, including iron trestle approaches.....	219 000
Cost of bridge floor.....	6 500
Allowance for freight charges on Atlantic and Pacific line.....	15 000
Total.....	<u><u>\$637 100</u></u>

"In the above estimate it is assumed that the 10 miles of steel rails and fastenings on the present line will be credited to the new line, but no allowance has been made for salvage on cross-ties and bridges on the present line between Powell and the Needles. Attention is called to the high estimate of cost of the caisson for the east pier, and we would not advise contracting it for this sum, believing that, if put down at cost and a percentage (the latter limited to \$12 000), the actual cost would be at least 10 per cent. below the estimate for this item. But desiring to cover all possible contingencies, we have deemed it best to make an allowance sure to cover the same. All of which is respectfully submitted.

(Signed) JAMES DUN,  
Chief Engineer,  
*St. Louis and San Francisco Railroad.*

(Signed) A. A. ROBINSON,  
Chief Engineer,  
*Aitchison, Topeka and Santa Fé Railroad.*"

Formal notice was sent by wire notifying the Phoenix Bridge Company to suspend work on the metal of the superstructure on the 16th of March, and on the 19th the writer received instructions, as indicated in this report, with further direction to determine the actual depth to rock at the caisson site. The contracts with the respective contracting firms having been modified to meet the changed conditions, and the first caisson broken up and removed to avoid its being buried and forming an obstacle to further progress of the work, the work was suspended so far as referred to the bridge.

It is proper to review the question of soundings here, as this experience by which the whole progress of the work was stopped and the completion of the bridge was delayed for nearly a year with the cost of maintaining the old line in the valley, impresses itself as one of the first importance. Going back to the time of making the design, while the soundings at the pier site were being made and about to be completed, the following instructions were telegraphed to Mr. Jenkins on the 25th day of December, 1888:

"Kindly arrange to have holes drilled to considerable depths, both on the site at Station 169 + 25 and the site at west edge of river 194 + 90 to see if any soft material in terposes."

On December 29th, Mr. Jenkins reports as follows:

"Herewith we hand you records of soundings made at Red Rock crossing; also profile of soundings at Station 189 + 25. In making these soundings we have been careful to report only what experience has taught us is the true 'bed rock,' or the proper foundation upon which to build, but as you well know, there is always room for variation in reporting soundings where the material composing the bed of the river is so varied in character as in the present case. On the west side bottom we are positively sure that the bed rock is of the same character as that seen outcropping near the water's edge, viz.: red breccia, very compact and hard, on the east side (*i. e.*, east half of river bed—*Rowe*) the bed rock is more irregular and not so readily distinguished, but is no doubt composed of two varieties of the concrete, the softer overlaying the harder stratum. We would recommend, therefore, for a pier say, at Station 189 + 25, that the excavation be made through the upper stratum of concrete to the lower or harder material in order to obtain a proper or safe supporting power. Although but few boulders were encountered in sounding (and those small ones and principally on the east side), it is always advisable to provide a special clause in the specification for boulders, particularly if pneumatic caissons are to be used in the construction of the pier foundation."

The following is Mr. Jenkins' record of the test drill at the pier site in question, sounding No. 23 at Station 189 + 25 in accordance with the telegraphic orders just noted :

Boring No. 23 at center Station 189 + 25 :

470.64 Elevation of water surface.

3.00 Water.

467.64 Surface of sand.

6.19

461.45 Gravel (or drift), boulders and sand.

0.61

460.84 Surface of breccia, red in color, hard and compact,  
5.63 but easily broken up with drill.

455.21 Hard stratum of solid substance, probably "breccia" of same character as eastern shore.

9.80 to bed rock—15.43 to harder stratum.

The tools used by Mr. Jenkins consisted of a 1½-inch drill, or what is termed the "jet drill," working in a 1½-inch casing, for which ordinary wrought-iron pipe is often used. Steel pipe, however, was provided as being stronger. The drill rod was made of three-quarter-inch pipe, and the jet was furnished by a double lever hand force pump. This outfit being light, was well adapted to use in the river by aid of a light flat-boat, anchored at the desired spot, and was found to penetrate compact gravel readily, even passing directly through boulders of 1½ feet diameter. Usually, however, it would either shift or turn the rock aside, or, owing to the elasticity of the casing, deflect to the most yielding side and so pass on.

Immediately on the discovery of the difficulty at the caisson site, heavier tools were made, 2 and 3-inch, and pending the decision as to what course would be decided upon, several holes were pushed down in and about the site. It was found that, while difficult to penetrate the first bed of boulders which were laid bare in the caisson, yet some hours of persistent labor would accomplish this, and the drill would proceed with little further difficulty until the depth of 64 feet was reached (elevation 406.00), when in every case the work would stop, either from breakage of the tools by long and persistent pounding or by the collapse of the casing.

In June following, heavier tools of much the same character were pro-

cured and placed upon a flat-boat; a No. 6 steam force pump, and an engine to run the machinery, were provided. The 3-inch tools were first used to the depth of 64 feet, before reached. The casing used was strong 3-inch wrought iron pipe, armed with a steel shoe to strengthen the end coming in contact with the work. Beyond this, however, progress was very slow and difficult; boulders were encountered at every move, and it is presumed, were in most cases cut through or broken, as they were imbedded in compact gravel cemented into a mass by a silicious cement, preventing any movement to allow the casing to pass. When the casing could no longer be driven, the 3-inch tools would be laid aside and a 2-inch casing and drill would be introduced inside of the 3-inch casing. This would be driven forward a foot or two until either the tools or the 2-inch casing would collapse; then withdrawing the 2-inch casing and tools, the 3-inch would be again resorted to. Thus foot by foot would be gained. The 2-inch casing, when withdrawn, would somewhat resemble the letter "S," becoming so distorted as to prevent the movement of the drill rod in it. During the whole progress from the first sand bed to the boulder bed, the jet would throw out a constant stream of clear, river washed sand, from which the cuttings of the drill were difficult to distinguish.

When, however, the boulder bed was entered, the cuttings became more plentiful, and the character of each stone passed through was as well indicated, almost, as if the boulder could be seen. Pure quartz, gneiss, porphyry, trap and the various types of volcanic rock and some very fair specimens of natural concrete were among the exposures (note at the time that this formation was afterward reached in sinking the caisson, bits of partly siliciurated wood were found above, on or near the boulder bed, which would indicate somewhat the great length of time this formation has occupied its present position).

Finally, on reaching 80.7 feet in depth, the sand thrown by the jet changed, or rather became highly colored and reduced in quantity, and then finally ceased, and well identified cuttings of the red breccia were obtained, identical with that outcropping on either shore. Thus, the first test hole was located at the 189 + 08, and 18 feet south of the center line of the bridge. Another was put down at the 189 + 50, about the same distance from the center line on the opposite side, of which it is unnecessary to say anything except that the same formation was reached; at  $\frac{1}{10}$  foot less depth; otherwise the history does not materially differ.

In each case the bed rock was penetrated about 1 foot. In both the casing was so contorted by its encounter with large boulders, as to prevent further progress without undue expense. Indeed, the formation was so thoroughly identified as to render it unnecessary, especially as the character of the material below the elevation 406, as developed by all the borings, forced the conviction that it would be very difficult, if not impossible, to sink a caisson below that point. There are the best of reasons for believing that a caisson sunk half the distance, or to the elevation of 436, would have been entirely secure, as from that elevation, the boulders were found so numerous and so compactly embedded in cemented gravel, that it was highly improbable that the river would ever, in the lifetime of the bridge, come to that depth at that point.

The lesson taught may be of value in the future and in this connection every hint should be heeded and every event scanned. The fact mentioned by Mr. Jenkins as to the change of character in what was supposed to be bed rock, from that well identified on the west half of river bed and that found on the eastward, it can be seen should have cast suspicion on the conclusion drawn, as well as the very moderate depth, at a point so distant from the east shore. Again, the conformation of the river, by close examination of the vicinity of the bridge, as well as the character of its drift, should have indicated the probable presence of the accumulation of drift; while the conformation for a mile or two above would have sufficiently indicated its ancient character. By examining the map of the bridge site, it will be readily understood that the tendency of the channel is westward at that point, while the rock bound bank on the west side of river for a mile or more above, with the further influence of a large cañon entering the river above the head of the Needles Cañon on the same side, with its frequent discharge of drift including boulders of considerable size, tend to crowd the river channel eastward. The deep scour of the river must occur at time of extreme flood, at which time its course is most in conformity with that indicated on the map, and it is hardly possible that it should attack the pier site.

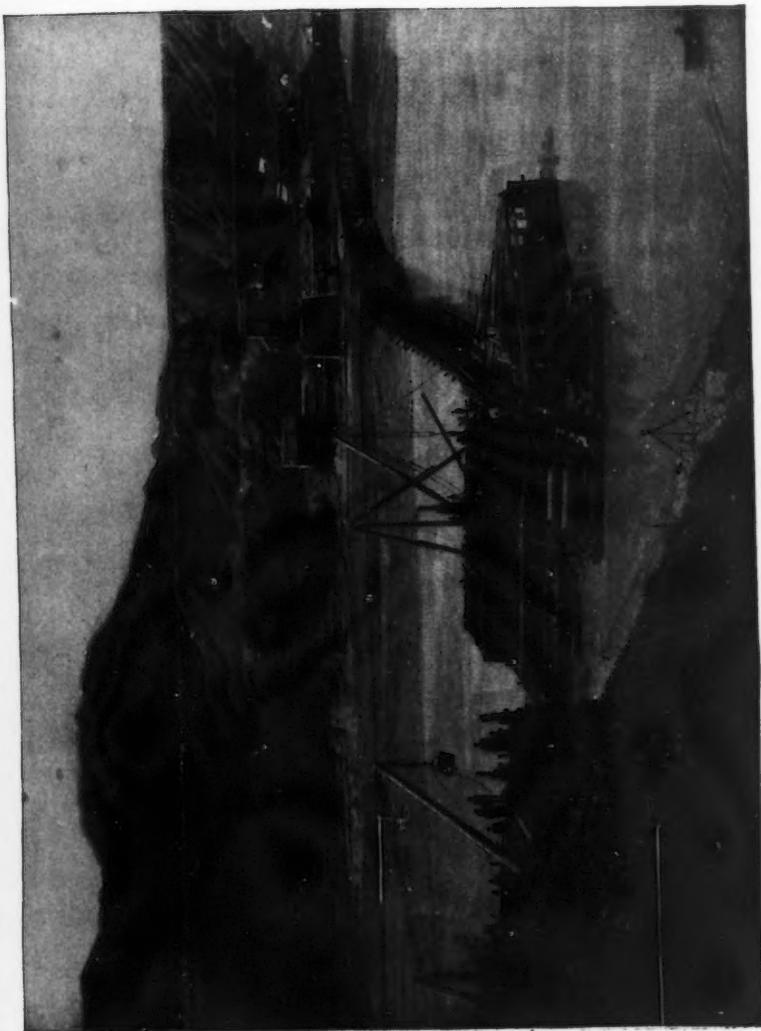
The soundings in the Colorado River were unusually difficult, both in consequence of the great depth, and the heavy, coarse character of the drift, to which were to be added the evanescent character of its quick-sands and the violence of its current. From all those combined with

the haste in which the work was prosecuted, it is only surprising that more mistakes did not result. Had the suggestion made at the time of making the preliminary report been followed, time and money would have been saved, as well as much embarrassment. The engineer must know what the conditions are before he can either plan or proceed with the erection of a structure of this kind with any degree of safety or credit. In any similar case larger and heavier appliances still than those last used would be safer, particularly when boulder beds are apprehended, as "well men" all know the fact that a formation of that character is the most difficult of all to penetrate.

For the benefit of others in the future, it is here stated that it was found that a water pressure of 18 to 20 pounds on the jet was the best; a heavier pressure would raise gravel and fragments of rock to such extent as to impede the free stroke of the drill rod. With softer rock, perhaps a less pressure would be necessary. The sands as well as the rocks in the Colorado River are nearly 15 per cent. greater in specific gravity than those of other localities farther east. And further it may be said, that in work of this character, extreme care must be taken to not overstrain the tools, thus incurring breakage or disconnection of the drill rod coupling. It was found that by changing drills and reconnecting the rods every five hours, work could be safely and steadily prosecuted, while any disregard of this precaution would result in a job of fishing, the nature of which every experienced drill man will appreciate.

The result as above having been attained in July, 1889, the material for the caisson was ordered and received, so that in the latter half of September, the work was again resumed; the tramway (Plate CXVII) by which the caisson site was reached was partially rebuilt, about 200 feet having been swept out by the high water of June; 20 feet depth of the river bed having been swept out and again replaced, as the river subsided. On November 21st, air was put onto the caisson and kept on continuously until February 11th, when the filling of the caisson was completed, resting on the boulder bed at elevation of 409.00. The sinking progressed steadily from the first, and the corners did not get out of place to exceed 4 or 5 inches either longitudinally or laterally, during the whole progress of sinking. The material proved even harder and more firm in character than was inferred from the boring. Owing to the unyielding character of the material surrounding the caisson, comparatively a small

PLATE CXVII.  
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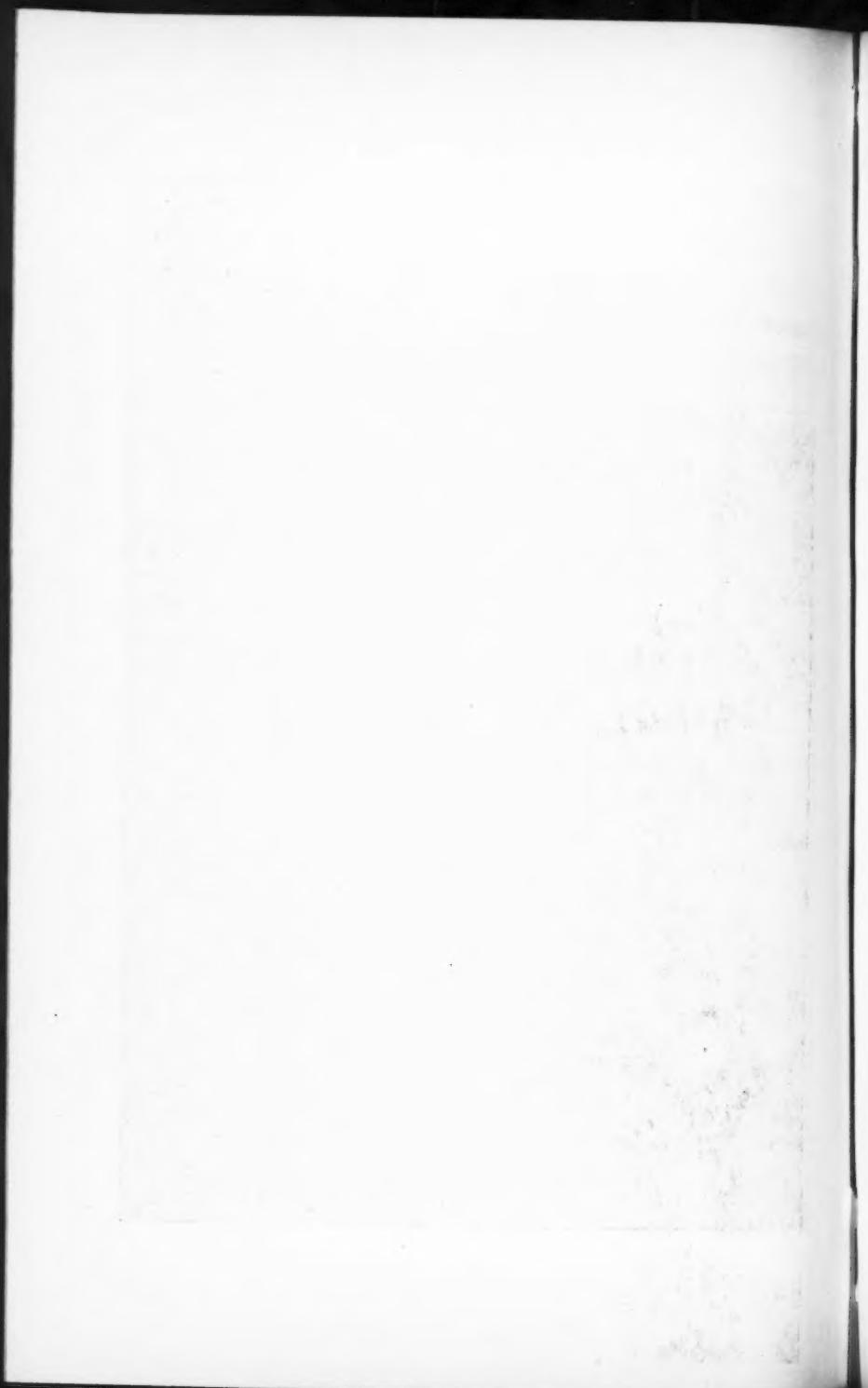
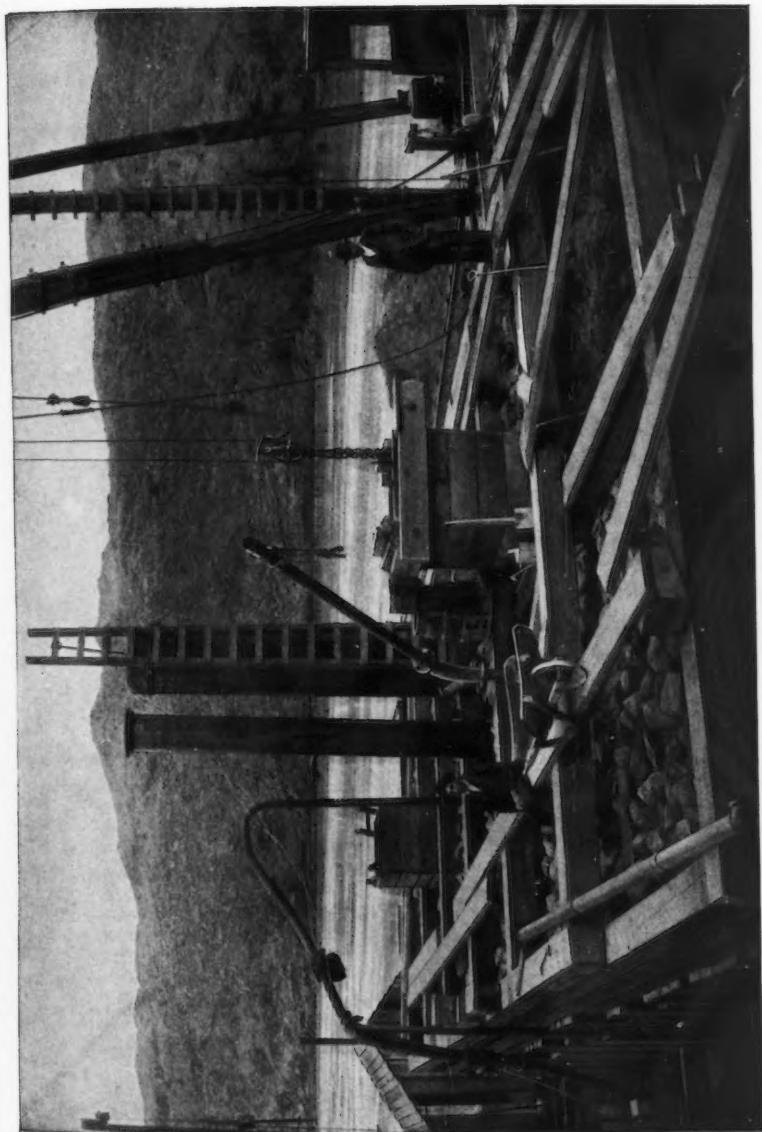


PLATE CXVIII.  
TRANS. AM. SOC. C. E.  
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RED ROCK CANTILEVER BRIDGE.





amount of excavation was made beyond its area. The caisson, a plan of which is shown in the photograph (Plate CXVIII), was carefully and strongly built, and measurements made at the time of sealing, showed that it had borne the strain with scarcely any distortion. The table appended gives a history of the progress in sinking.

To determine in advance just when to stop the crib and commence the footing courses of the pier, was very difficult. In order to reach elevation 406 required that a portion of the pier should be built, as the great distance through which the caisson had already sunk through rough, unyielding material caused great friction on the sides of the crib, and made it necessary to withdraw almost or quite all the air pressure to move it. It was decided to stop the crib at a height that would put it about 4 feet below low water, so that the timber would not be exposed; but at the same time, to crowd the caisson down to 406 or possibly 2 feet more to 404. To do this, it was found necessary to put on all the footing courses and about one-half of the neat pier, and afterward lay temporarily two additional courses. Notwithstanding this, the caisson failed to settle beyond 409. After repeated trials to cause it to do so, and after making a thorough examination of the foundation, it was decided to excavate from 2 to 2½ feet all around under the cutting edge, which could readily be done, the wall standing perfectly firm, and follow with the beton in sacks until it reached the cutting edge at every point, which was done. The center was then excavated and sealed, followed by filling the working chamber. The caisson was already about 5 feet into the boulder formation at the northeast, 3 feet at the northwest, 2 feet on the southeast, and down to it on the southwest corner, indicating that its surface sloped to the southwest. The excavation made and replaced with "beton" fully reached the firm foundation at all points, and over the whole extent of the chamber. The boulders formed the greater mass of the underlying stratum and were bedded compactly, so that in excavating, each separate stone had to be picked or pried loose. While the chamber was being sealed there was a steady daily subsidence of the caisson of about .05 feet, but when the beton reached the cutting edge, this ceased and no further settling occurred up to the time of completion of the bridge. This movement was utilized to right a slight tilt of the pier to the west by sealing that side first. The slight remaining tilt was taken out in the subsequent courses of masonry and the pier finished systematically, although slightly larger

than designed. It might have been possible to spend a larger sum of money in trying to force the caisson lower, but the author doubts if it was possible to do so, and no good end would have been subserved. The top of the crib is about at the level of extreme low water.

Of the material used, each part will be treated in its turn. A record of the progress of the work of sinking the caisson is given in Appendix "A." The timber used for the caisson and crib consisted mainly of what was termed Oregon pine (yellow fir), a most excellent timber, strong and firm, though somewhat coarse grained, which weighed 35 to 40 pounds per cubic foot when well dried. The amount used in the different parts was about as follows:

	Cubic Feet.	Feet, Board Measure.
Working chamber, including 3 inches inside casing.....	6 880	= 82 560
Roof, 8 feet thick .....	12 992	= 155 904
Crib, including 3 inches casing outside .....	20 071.5	= 240 855
Making total amount of timber (neat)....	39 943.5	= 479 319
Iron bolts, stay and drift and steel spikes 29 tons = 58 000 pounds.		
Concrete (beton) put into crib, 47.7 cubic yards per running foot .....	2 290	cubic yards.
Concrete (beton) put into working chamber....	580	"
Total.....	2 870	"
weight at 4 050 pounds per cubic yard, saturated.		

Timber at 35 pounds will absorb water to nearly 80 per cent., so that the timber at 35 pounds when dry, and absorbing 80 per cent. ( $= .8 \times 35 = 28$ ), will weigh  $35 + 28 = 63$  pounds per cubic foot, or have about the same weight as water. Then taking the caisson at the time it stopped we have, 10 800 square feet of surface on which the pressure of the material outside tended to produce friction; the excess of the weight of the concrete in the crib over the displacement of the water, which, being 101 203 cubic feet, at 62½ pounds per cubic foot, weighed 3 162 tons. At this time there were also 125 cubic yards of masonry on the crib, equal to about 800 tons. Therefore, we have—

2 290 cubic yards of beton in crib = .....	4 637.25	net tons.
125 cubic yards of masonry.....	804.00	"
Total.....	5 441.25	"
Resisting this is the uplift of the water.....	3 162.00	"
Leaving the downward pressure of.....	2 279.25	"

PLATE CXIX.  
TRANS. AM. SOC. C. E.  
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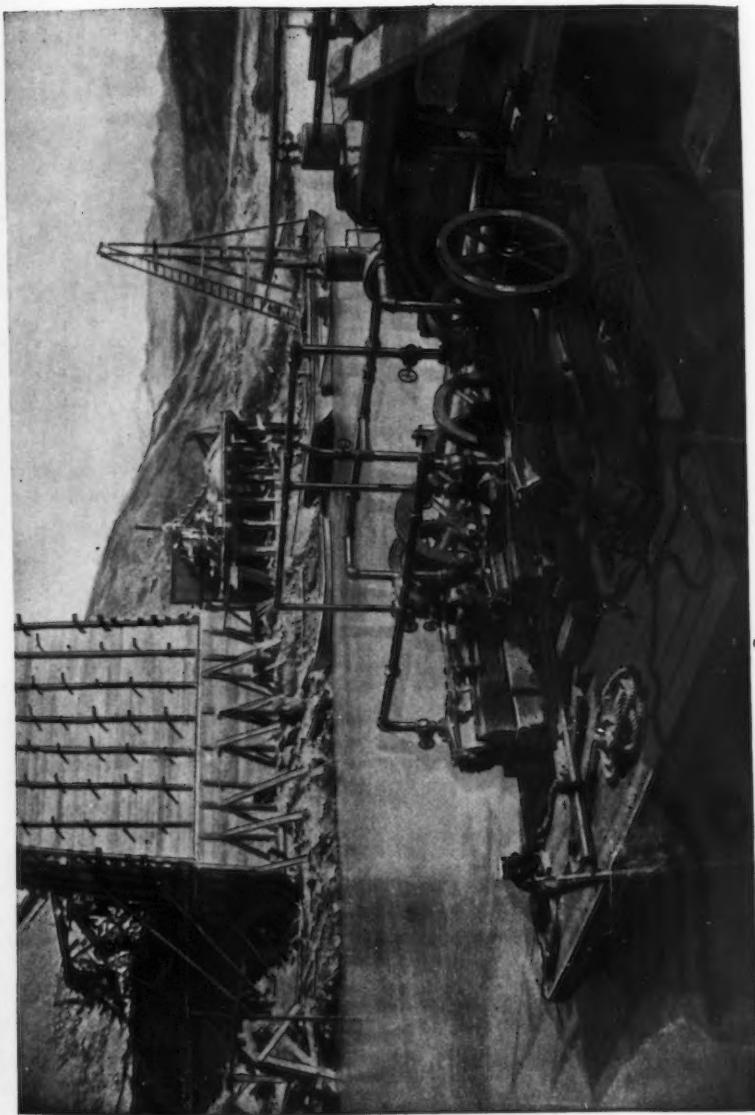
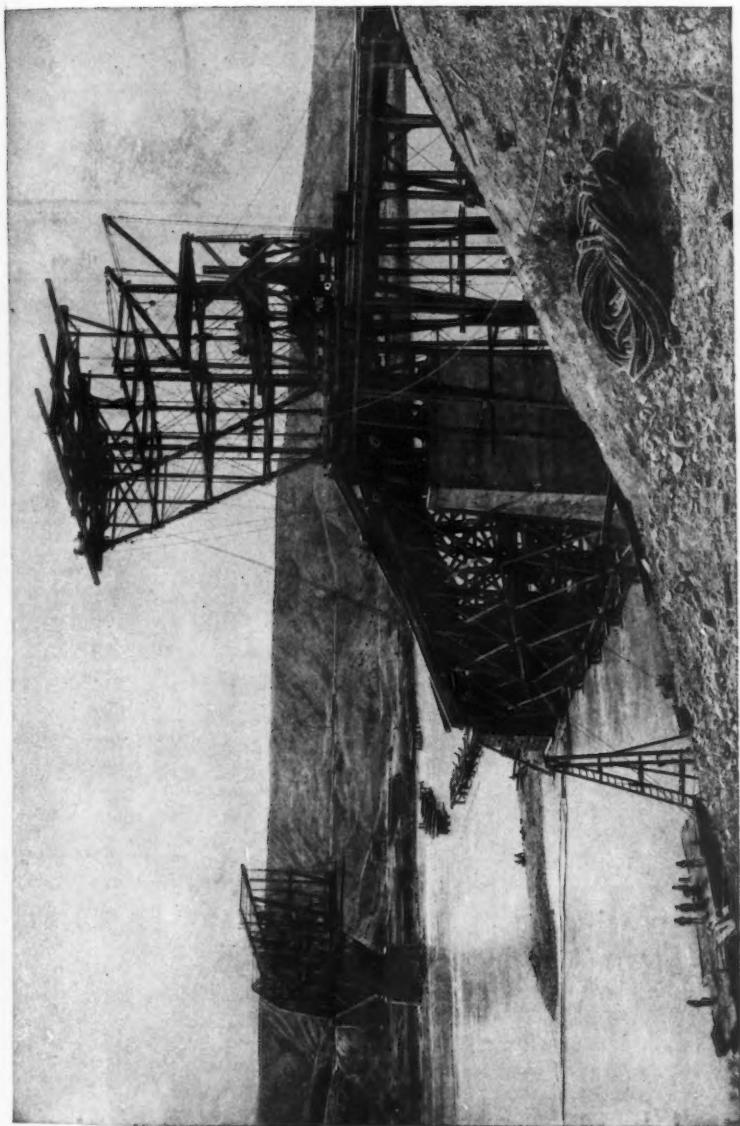




PLATE CXX.  
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equal to about 420 pounds per square foot when the air pressure is entirely off. When the air was on at 27 pounds per square inch, this tendency downward was entirely overcome. The air was entirely and repeatedly removed toward the close, and much of the surrounding material drawn. In ordinary sand and gravel the caisson could have been pushed to any depth desired. It is presumed that boulders were crowding between the pier walls of the excavation and the sides of the crib to such extent as to sustain it, notwithstanding the sudden release of the air, causing the sudden application of the 2 000 tons as before stated.

The whole resistance was below the elevation of 440.00, where the material was all of hard, firm gravel and boulders. The following will give much information as to the relative cost of sinking, taking labor roll alone:

November, 10 days, total roll.....	\$2 611 72	Depth made 9.94 feet, mean cost per day \$261.18, per vertical foot.....	\$262 75
December, 31 days, total roll.....	10 026 05	Depth made 23.38 feet, mean cost per day \$323.42, per vertical foot....	428 83
January, 31 days, total roll.....	10 710 50	Depth made 26.80 feet, mean cost per day \$346.83, per vertical foot.....	*400 00
February, 11 days, total roll.....	3,759 05	Filling chamber 11.5 feet, mean cost per day \$341, per vertical foot...	326 00

The less amount of cost per foot for January was owing to the use of the air pressure for raising much of the material during that month. The pressure used reached 27 pounds, which was maintained during the time of filling the working chamber. The air plant consisted of three compressors, two of which were double cylinders 16 x 24 inches, and one 12 x 18 inches. Two were used while excavating and one held in reserve. These were driven by two 75 horse-power boilers and one of 50 horse-power. The whole air-plant was placed on a boat 24 x 60 feet, built for the purpose (Plate CXIX), and all housed in to protect it from the sand storms. Fuel (coal) was unloaded from the car in reach of the compressor. Water was drawn from the river, and settled in a submerged flat boat as far as possible, but was always bad, being destructive to the boiler flues and causing much trouble. The 25th day of February saw the substructure complete and the west half of the superstructure nearing the center point of the bridge (Plate CXX).

The stone used in the piers, coping and bridge seats was obtained at the Chino quarry, about 140 miles east of the bridge site, in Arizona,

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\*The cost of the last foot or two of sinking was nearly \$2 500 per foot.

where there is an extensive ledge. The color of this stone is a light, clear, cherry color, free from stain, and, laying as it does in thick ledges, it can be quarried of almost any size and shape desired. It is a compact sandstone, very hard and strong, homogeneous in texture, weighing about 160 pounds per cubic foot ( $\frac{4}{3} 220$  pounds per cubic yard), and cutting a little easier than granite. A block of the stone  $3 \times 3 \times 6$  inches was tried by Prof. Waddell, at Kansas City, and remained unbroken after being subjected to 12 000 pounds per square inch (the extent of the capacity of the press). In the quarry the courses run from 20 inches to 6 feet, but owing to its homogeneity the stone can be split into any dimension or shapes desired. In constructing the two main piers many of the headers overlapped at the center of the pier where the pier was 15 feet thick, and some of the stretchers show 9 feet in length on the face of the pier. The courses varied from 26 down to 18 inches in thickness, with coping 22 inches. The main seats for the four pedestals carrying the whole weight of the bridge are 9 feet square and  $2\frac{1}{2}$  feet thick, made up, however, of four sections each. It was first designed to use stones of full size, but in consideration of the excessive cost and difficulty of handling and placing, without much heavier appliances than were at hand, it was decided to use the latter.

	Pounds.	
The anchor arm, including floor, weighs	593 900	
The cantilever arm, including floor, weighs.....	613 100	Then as there are two seats on each pier $\frac{3943550}{2} =$
The two bridge seats, pedestals and piers .....	79 850	
		1 971 775 pounds on each seat. Each seat $7 \times 7$ feet = 7 056 square inches, and $\frac{1476775}{7056} =$
		7 056 pounds = 279.4 pounds per square inch, or a little over $\frac{1}{4}$ of the ultimate crushing load.
The suspended span, one-half including reaction of anchor.....	656 700	
Live load on all at 3 000 pounds per lineal foot.....	1 980 000	
Making a total weight in pounds. 3 943 550		

The two main piers (Plate CXXI) and the abutment and pedestals supporting the viaduct were built of this sandstone, as well as the coping of the two anchor piers.

The anchor piers (Plate CXXI) were built of beton, the specifications for which will be found in the Appendix E. The west anchor pier was built with hand mixed beton, as well as the foundation for the east anchor

pier and the west main pier. At the resumption of work in October, 1889, however, a mixing machine run by steam and having a capacity of 150 cubic yards in ten hours, was used, not only economizing in labor, but giving better work. The Gillingham English Portland cement was used and the specifications varied somewhat. The concrete for the east anchor pier, caisson crib and working chamber were provided with a composition as follows: The mixer being arranged with carriers to measure each ingredient, was arranged thus: Four cups cement, thirteen cups sand, and sixteen cups broken stone, carried up and discharged into the trough of the machine. A minimum of water was discharged into the sand and cement so that they became mixed before receiving the broken stone. The water was so gauged that it should not appear when the concrete was placed in the work and well tamped. The concrete was discharged into wheelbarrows or one-third yard iron buckets and conveyed to the work. Meantime, a quantity of one-man stone was distributed over the work to an amount of perhaps one-half of the amount of the concrete, and the latter driven around, between and over the stone. It was required that the broken stone should be well wetted before feeding into the machine, as well as the stone used in the work, so that thorough contact with the mortar should be secured. It was also required that each course in the pier should be well wetted after the tampers had gone over each portion of the work, to the end that all crumbs of mortar or dust should be smoothed down, and a clean surface should be secured, to give more perfect bond to the subsequent course. To prevent possibility of the feed of cement being interrupted, an inspector was placed at the machine to guard against this as well as to govern the amount of water used.

So important is this, and the frequency that it will occur, that it would be well to so arrange the feeding hopper, especially of the cement, that it would feed itself from a hopper holding a considerable quantity. The importance of having the proper amount of water and no more, will be understood when it is considered that to the packing quality of the concrete, its superior strength as a beton is due, as well as its less tendency to shrink. It is well known that, if well packed, perfect adhesion to the stone with which it comes in contact will be secured. If imperfectly packed from having too much water, it will shrink and break its bond. In the main piers the same concrete mixture with a slightly larger allowance of cement, omitting the "one-man" stone, was used.

First laying the face stones of the course and filling and ramming the face seams with mortar ; the backing was all so laid as to leave room for ramming between ; then the proper quantity of concrete was mixed and it was so deposited as to secure thorough packing, in several courses, until the whole course was leveled up ; not forgetting to first wet all the stone with a sprinkling pot. It is not doubted that the pier will have all the stability of a monolith; the course pursued is much preferable to the practice of using drowned mortar and spawls, and the concrete will be less likely to separate than even well jointed solid courses.

The cement, of which nearly six thousand barrels were used, was tested by taking numerous briquettes from the quantities mixed by hand, and by the machine, and were found so uniformly good that the only precaution used was to break the briquettes from several barrels together, thus avoiding the bad effect from an occasional barrel that might fall below the standard. At a time before commencing the work, enough tests of neat cement were made to indicate a surprising uniformity in this brand. The test was usually made on ten or twelve briquettes each day, taken from the machine and allowed to dry in a shaded place twenty-four hours ; then immersed in water for the same period, and then immediately broken. The testing machine was the "Fairbanks," and the briquettes gave a tensile strength from 160 (exceptional) to 225 pounds per square inch, a mean of nearly 220 pounds. Samples kept thirty days gave over twice that amount (410 pounds mean); some gave over 500 pounds. It was early found that the strength was governed largely by the character of the sand. No drift (wind) sand was used, as it was found to give very low tensile strength, but a bed of clean, water-washed sand was found convenient, having quantities of fine gravel intermixed, and later, a large bed of coarse sand in the bluff east of the river, so well adapted to the work as to justify its shipment around nearly 30 miles, to use in the caisson. Particular attention is called to this sand as worthy of shipment a long distance, for public buildings and other important work. A comparison of tests shows that this sand gives 30 per cent. excess of strength over good river sand (525 pounds at thirty days).

The broken stone was at first supplied from the débris of the Chino Quarry and from the volcanic rock found in the vicinity of the bridge, but it was found that the broken volcanic rock with which the "mesas" were strewn, could be collected at less cost, and being of the

same character, was substituted in the caisson work at a saving of nearly \$1 per cubic yard. The process of gathering was to rake these fragments of stone into windrows and haul them by wagon to a pile where convenient to load into a car when needed. An inclined screen was erected to separate the dust from the stone while conveying it to the car. Indian labor was used very successfully for this as well as for labor about the caisson.

The criticism has been made that the anchor piers should have been made with masonry rather than concrete. In reply, it is answered that, while making the original design, the concrete was considered most suitable and cheapest. Masonry of the character required would have cost \$20 per cubic yard, excluding cement, owing to the complicated structure, it having twelve angles instead of four as in an ordinary solid pier. The comparison would run thus:

Masonry, per cubic yard.....	\$20 00
One-half barrel cement.....	2 00
Freight on stone, per cubic yard..	3 00

Making total.....	\$25 00 per cubic yard.
Concrete, contract price.....	\$12 00
One and a quarter barrels cement	5 00

Making total.....	17 00	"	"
-------------------	-------	---	---

Making a difference of.....	\$8 00	"	" in favor
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of the concrete, and (there being in the two anchor piers 1770 cubic yards) a saving of \$14 160. Taking into consideration its better adaptation to the purpose, the criticism is answered. These, at least, are the considerations which impelled its adoption.

The anchor piers have not less than 700 cubic yards of concrete over the anchorage, which at 4 050 pounds per cubic yard, will give a weight on the anchorage of 2 835,000 pounds.

The uplift will not exceed the following:

Reaction of excess of weight of cantilever over anchor arm..	19 200 pounds.
" " half of dead load of suspended span.....	328 350 "
" " live load on cantilever arm 165 feet $\times$ 3 000 pounds	247 500 "
2	
" " live load on suspended span 330 feet $\times$ 3 000 lbs.	495 000 "
2	
	1 090 050 pounds.

(The heaviest train obtainable so far, that made up of coal, does not exceed 2,000 pounds per linear foot.) This does not take into consideration the cohesion of the concrete, which is greatly in excess of the simple weight of the mass. It would not do, however, to rely upon this altogether in any case, as notwithstanding the utmost care, the strength at the joints between courses will be less strong than the body of each course. As the anchor metal is so interwoven with the mass of the base of the pier, the above weight must be ample.

**LOCATION OF PIERS.**—As a supported tape measurement across the river was impracticable, the pier centers had to be located by triangulation. A base line 850 feet long on the west bank of the river was carefully measured, and from that two permanent monuments were fixed on the center line, one being on each bank sufficiently distant to remain undisturbed ("A" and "C," Plate I). This line was measured with two 10 feet iron poles furnished by the Phoenix Bridge Company, and correct at 70 degrees Fahr. temperature. Later a tape was drawn across, supported partly by the tramway and the balance by the suspended stretcher 100 feet long, confirming the triangulation within  $\frac{1}{10}$  foot. Then again, during the suspension of work in the summer of 1889, a large sand bar formed on the west side of the river, rendering it practicable to secure a good base line perpendicular to the center line of the bridge from the west main pier, which was done. Much care was taken and repeated measurements made at different temperatures, and they were found after applying the correction (.00000663 per degree Fahr.) to agree within  $\frac{1}{10}$  foot in 702 feet, the base being extended so as to use a parallel base 42 feet from the center line of the bridge. From this an angle of 45 degrees was carefully and repeatedly turned and reference points fixed at such a distance as allowed a clear sight over the east pier when finished. By these reference points the bridge pedestals were set on the east main pier, and the bridge erected. When the bridge erection met at the center of the river, the lower chord bars overlapped 5½ inches, and pins were inserted and quickly driven, as soon as the lower erection wedges were slightly slackened off.

**COMPLETION OF BRIDGE.**—The bridge was completed on the 25th of June, 1890, in a manner creditable to the Phoenix Bridge Company, the contractors, every part going together without a hitch and with commendable speed. It was, however, opened for the passage of trains on the 10th of May, by the removal of one engine and the floor

of the traveler, this being necessitated by the long expected disaster to the old line, in spite of strenuous efforts to maintain it by the Road Department. Within two weeks, too, the old bridge lost a pier and two spans.

The shop inspection was placed in the hands of Hildreth & Nettleton, and owing to the dissolution of that firm was carried through by R. W. Hildreth & Co. Later, the senior member, R. W. Hildreth, acted as Inspector of Erection. To avoid useless repetition, the report of R. W. Hildreth & Co. is given partially as follows:

(Copy.)

#### "INSPECTION OF RED ROCK BRIDGE.

##### "MATERIAL:

"The material used in the manufacture of the superstructure of the Red Rock Bridge was wrought-iron, with the exception of the working section of the main truss members, which was of open hearth steel. The viaduct approach was entirely of wrought-iron.

"The material purchased by the Phoenix Bridge Company of the following rolling mills: Phoenix Iron Company, Phoenixville, Pa.

"All angles, squares, rounds and flats not over 12 inches wide, Pottstown Iron Company, Pottstown, Pa.

"All wrought-iron plates over 12 inches wide, Charles Huston & Sons, Coatsville, Pa.

"Steel filled iron plates for anchorage girders, Pennsylvania Steel Company, Steelton, Pa.

"Open hearth steel blooms to be rolled by Phoenix Iron Company into shapes and flats under 12 inches wide, Midvale Steel Company, Nicetown, Philadelphia, Pa.

"Steel pins, Carbon Iron Company, Pittsburgh, Pa.

"Open hearth steel plates over 12 inches wide and steel flats for eye-bars.

"Rolling was started at Phoenixville and Pottstown in the latter part of January, 1889, and continued intermittently at the various mills until March, 1890.

"The requirements for steel being for three-quarter round test specimens, rolled from special ingots cast with each heat, are for the purpose of insuring the proper quality of metal. A number of tests from finished steel were required by the specifications, to serve as a check of the heating and rolling. The requirements for both iron and steel, while necessitating good material, were easily met by the rolling mills; we were, therefore, obliged to make very few condemnations on the results of tests.

"At the Carbon Iron Company we were obliged to take all tests from the finished plates and bars, as these mills had no facilities for rolling rounds. This was preferable, however, as it not only gave the quality of the material as well as the three-quarter inch round would have done, but also showed whether or not the steel had been injured in the subsequent process, as of heating and rolling, more thoroughly than the specified number as taken on shapes, etc.

"For every specimen of steel taken for a tension test, a similar one was taken for bending. In all cases these specimens were successfully bent, cold, 180 degrees to close contact. Considerable material was

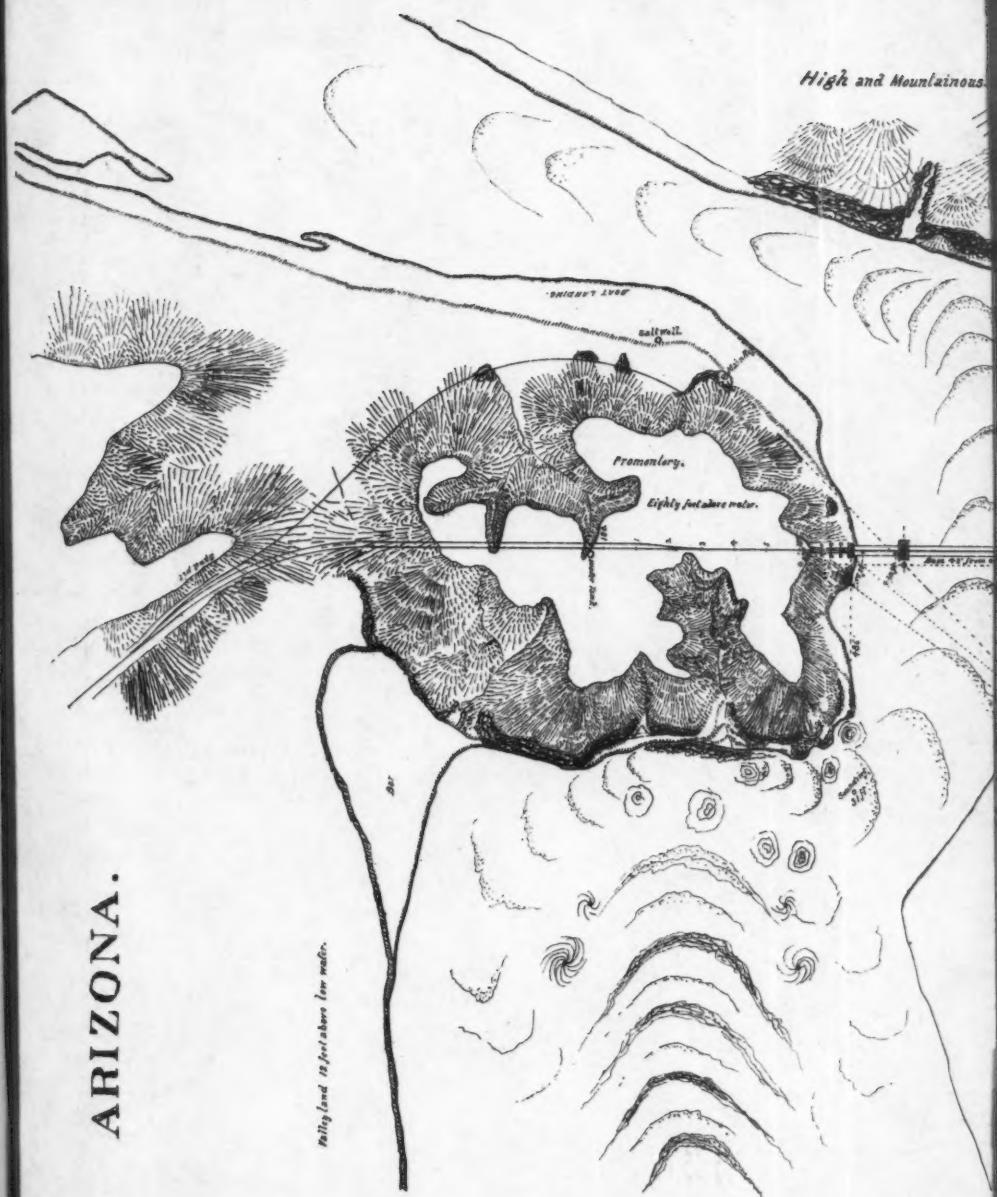
condemned at every rolling mill; this was rarely, however, for poor quality, but on account of surface defects and flaws.

"Tests of full-sized eye-bars were all made on the 600-ton hydraulic testing machine at the Athens shop of the Union Bridge Company. Only two of this series of tests broke in the head, and, in each case, this was due to the flaws discovered during inspection, the bars being selected for tests on that account. The result of the test of the  $5 \times \frac{1}{8}$ -inch bar was good, considering that the fracture was in the neck and caused by a flaw, so no retest was ordered. The retest of the  $8 \times 1\frac{1}{8}$ -inch bar gave good results for a bar that had already been strained so much beyond its elastic limit. These tests not only showed that the proportions of the heads were sufficient to break the body of the bar in all cases unless weakened by flaws, but that the material of the bars had not been injured by the manufacturing process to which it had been subjected at the bridge shop." (See Appendix "F.")

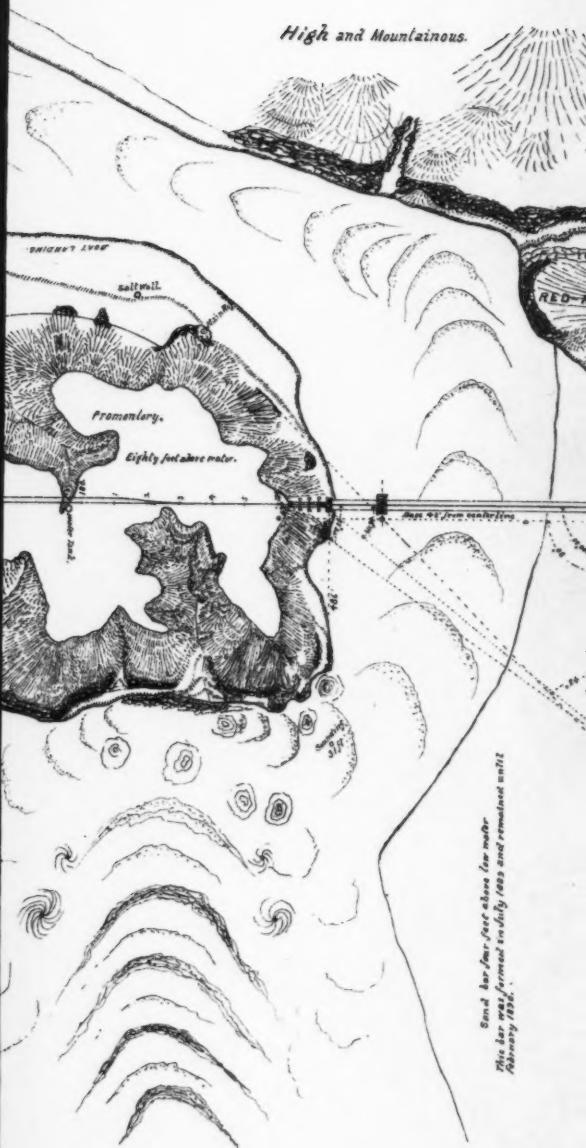
There was no formal test of the bridge after its completion owing to the difficulty of securing engines enough to produce the typical strain. As a matter of interest to all engineers, this should still be done, if practicable. Two 91-ton engines and a train of coal cars fully loaded (Plate CXXII), sufficient to cover the bridge from one main pier to the other, estimated to be about 66 per cent. of a full test load, gave a total depression at the center of  $3\frac{1}{2}$  inches.

In concluding this report the writer desires to give credit to all those connected with this important work. Mr. C. W. Smith, General Manager; A. A. Robinson, Chief Engineer on the part of the Atchison, Topeka and Santa Fé Railroad Company, and James Dun, Chief Engineer on the part of the St. Louis and San Francisco Railroad Company, advisory engineers, throughout its construction; Prof. S. W. Robinson, Professor of Mechanical Engineering, Ohio State University, Consulting Engineer; Martin Rapp, and W. F. Behrens, Assistant Engineers; R. W. Hildreth, shop and field inspector; each and all have the thanks of the writer for their efficient aid. The designers, Prof. William H. Burr and Prof. J. A. L. Waddell, the Phoenix Bridge Company, and Sooysmith & Co., contractors, each should share the credit of a good job well done. The accompanying views were secured by the aid of F. E. Evans, of Oakland, California.

# ARIZONA.



### *High and Mountainous.*



Sand bar four feet above low water  
This bar was formed in July 1893 and remained until  
January 1896.

PLATE CXI.  
TRANS.AM.SOC.CIV.ENGRS.  
VOL.XXV. N°518.  
RED ROCK CANTILEVER BRIDGE.

C O L O R A D O  
R I V E R.

**PHYSICAL MAP OF -**

17

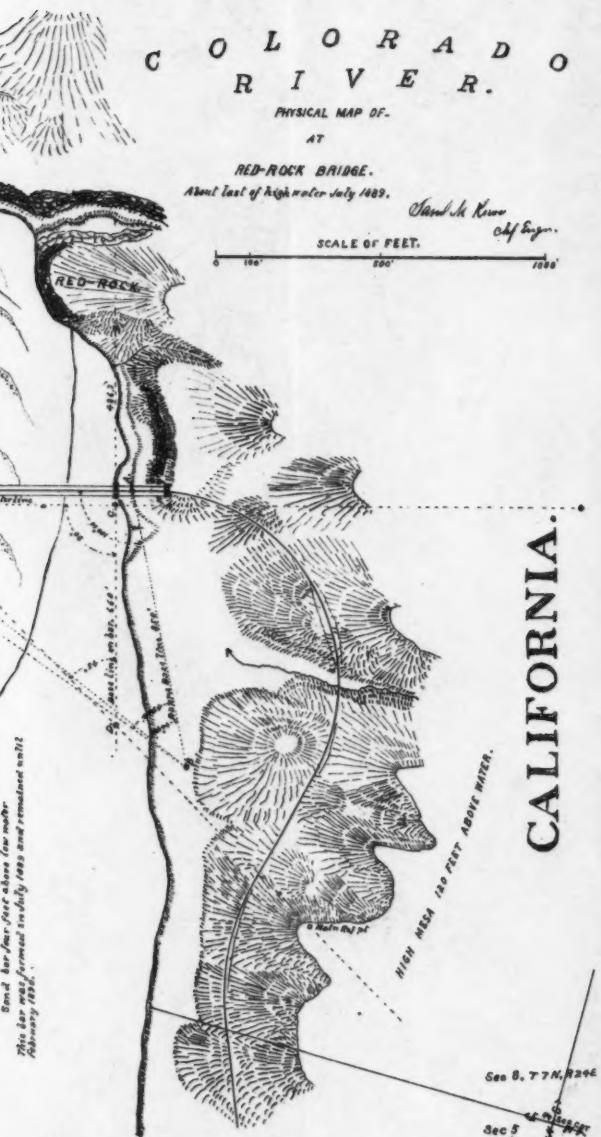
**RED-ROCK BRIDGE.**

About last of high water July 1889.

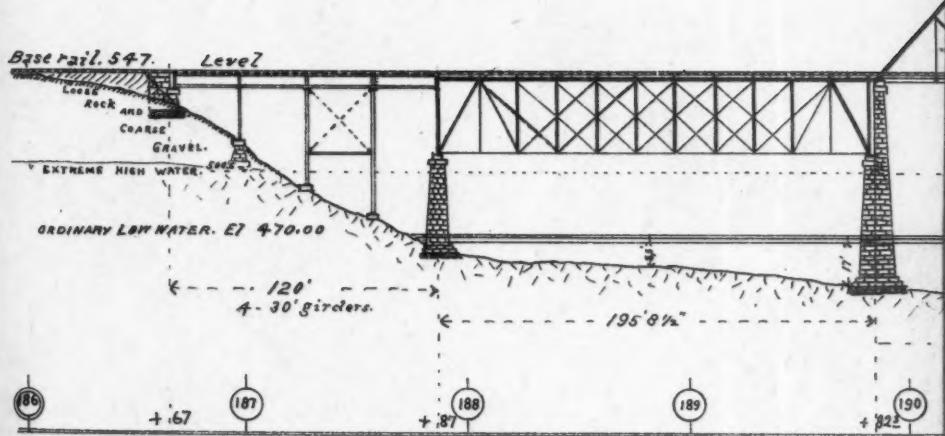
Sandie Kress

of Engen.

SCALE OF FEET.







RED ROCK CROSSING  
OF THE  
COLORADO RIVER.

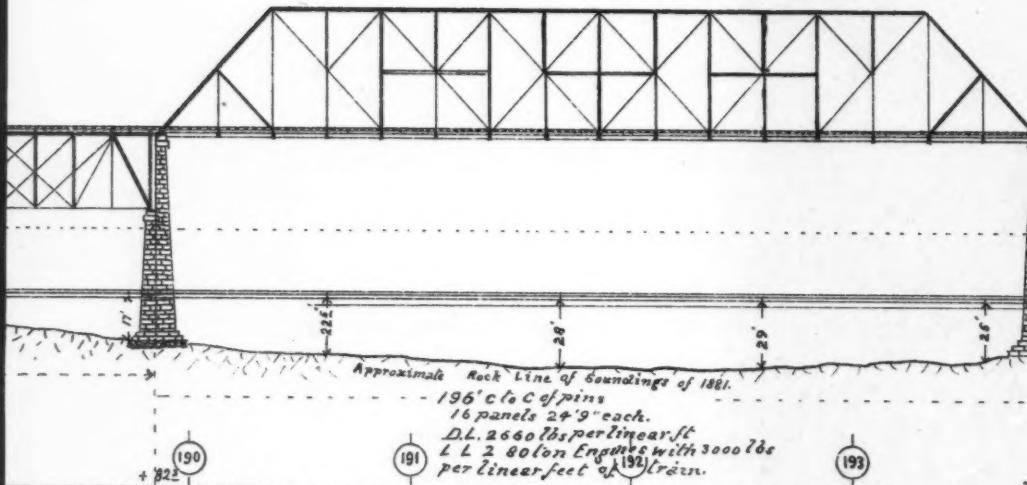
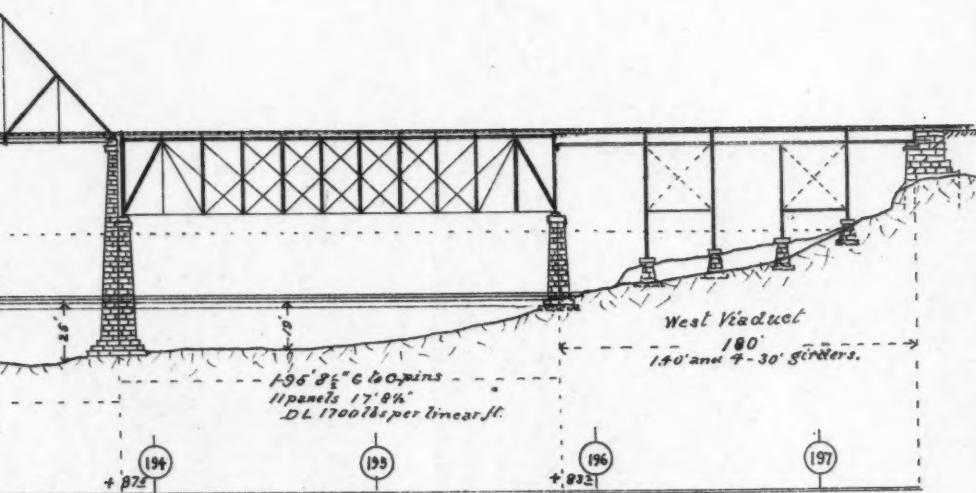


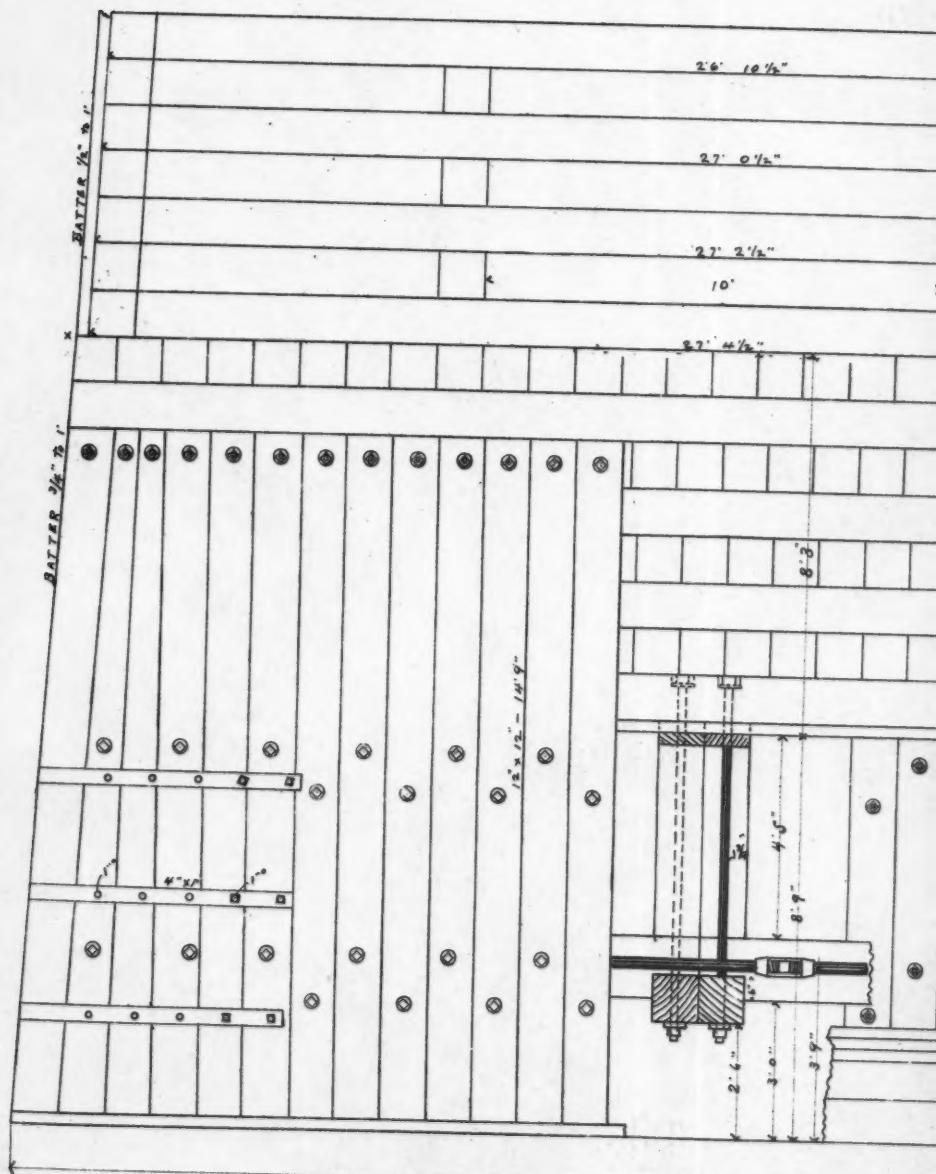
PLATE CXII.  
TRANS. AM. SOC. CIV. ENGR'S.  
VOL. XXV. NO 518.  
RED ROCK CANTILEVER BRIDGE.

KEYSTONE BR Co, Design, Modified.



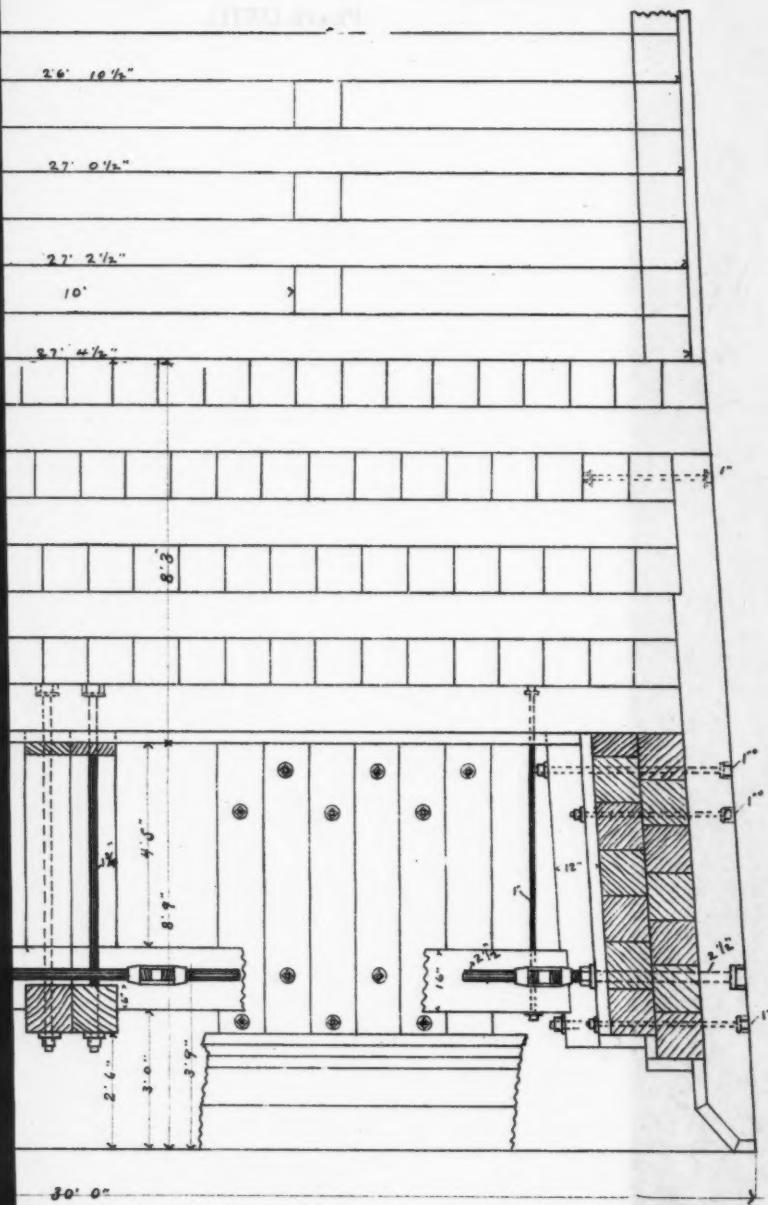
Sept 6<sup>th</sup> - 1888 *Saul M. Howe* EE:





CAISSON FOR COLORADO RIVER CROSSING

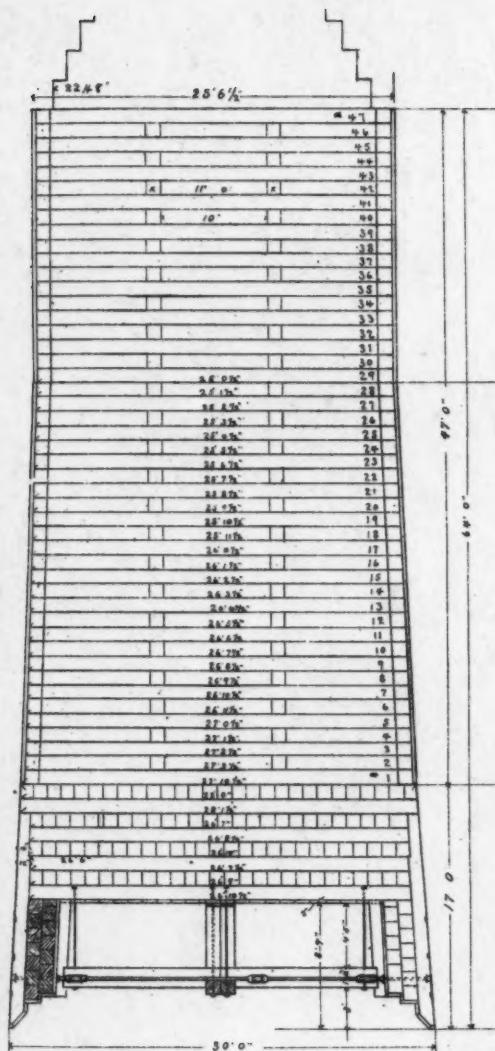
PLATE CXIII.  
TRANS. AM. SOC. CIV. ENGR'S.  
VOL. XXV. NO 518.  
RED ROCK CANTILEVER BRIDGE.



RIVER CROSSING







CAISSON FOR  
COLORADO RIVER CROSSING

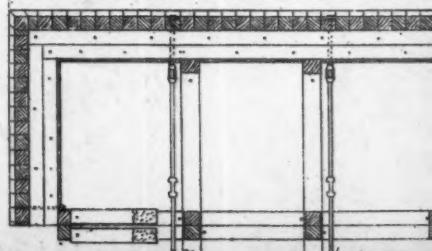
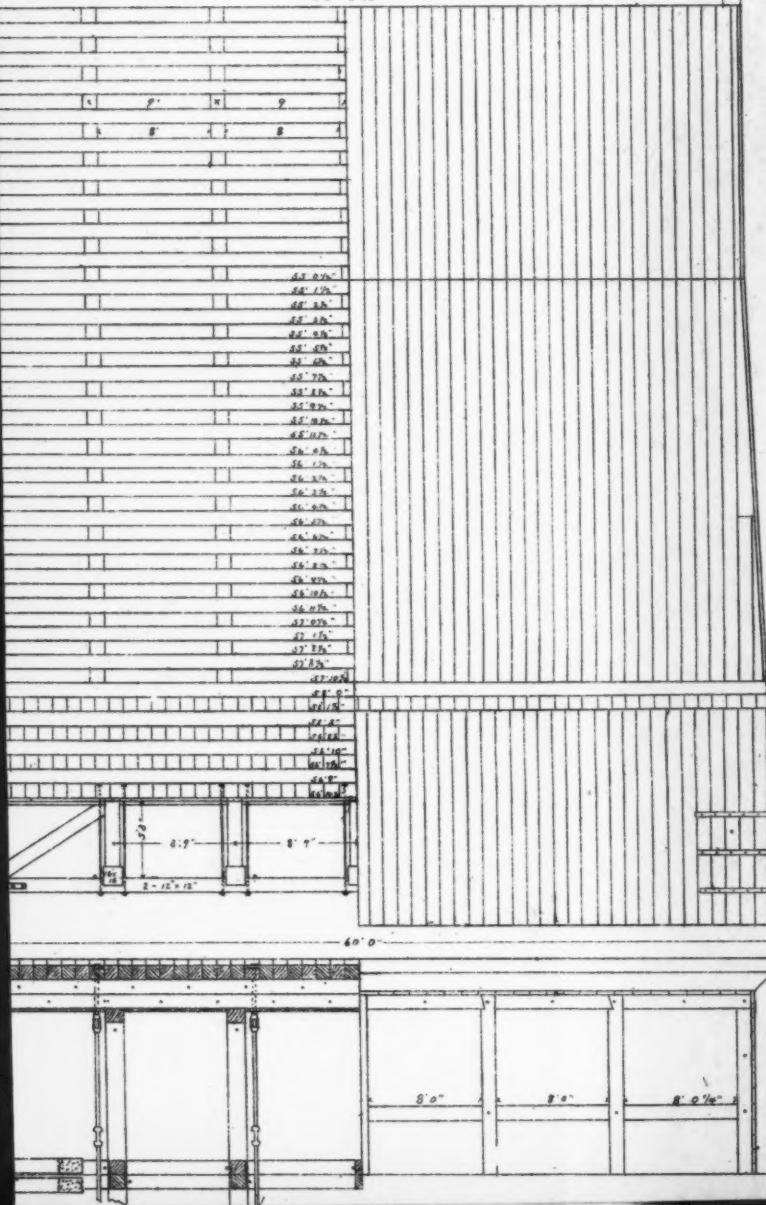
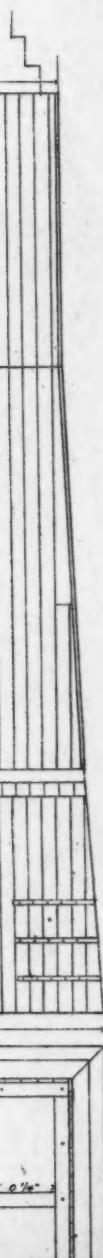
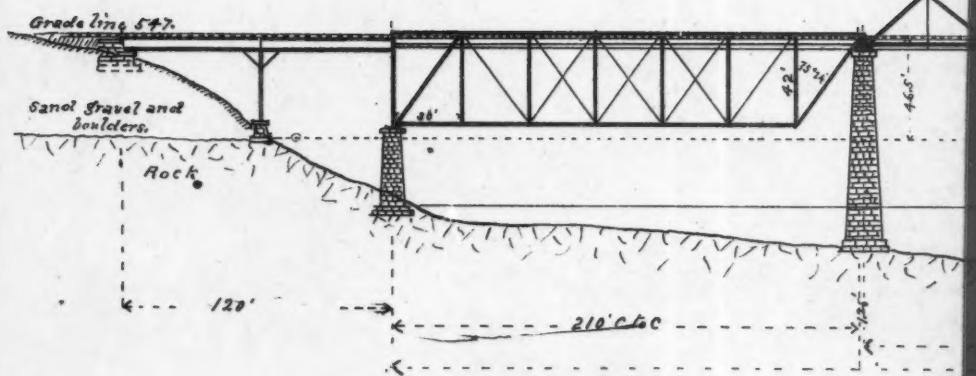


PLATE CXIV.  
TRANS. AM. SOC. CIV. ENGR'S.  
VOL. XXV NO 518.  
RED ROCK CANTILEVER BRIDGE.

55:6 1/2"

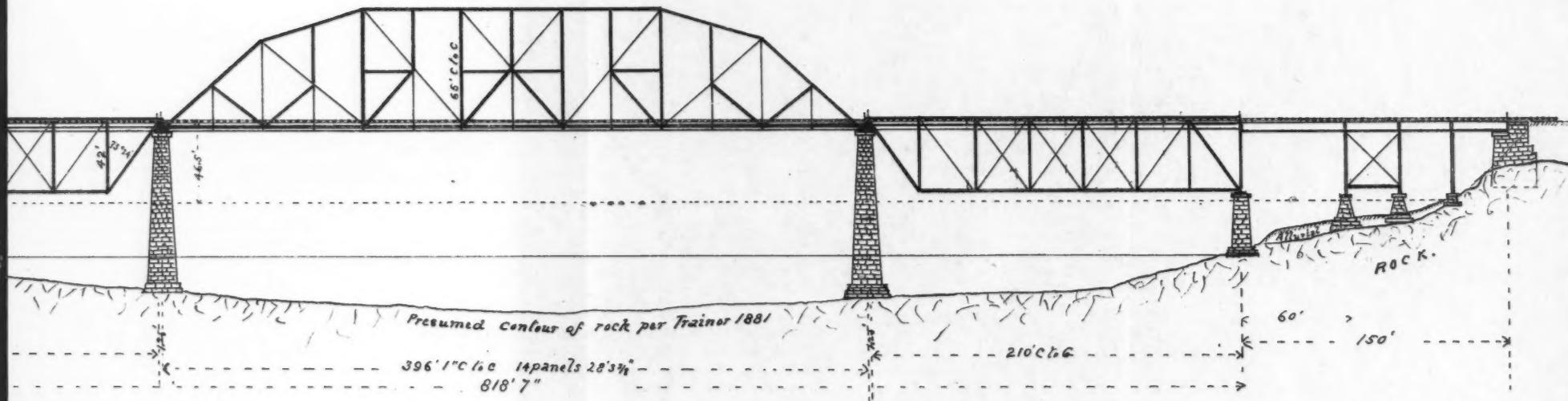






RED-ROCK CROSSING  
OF THE  
COLORADO RIVER.

P.B.C.o.s Design, Modified.



Sam M. Rowles,  
Albuquerque Dec 16<sup>th</sup> 1888.  
C.E. Eng.

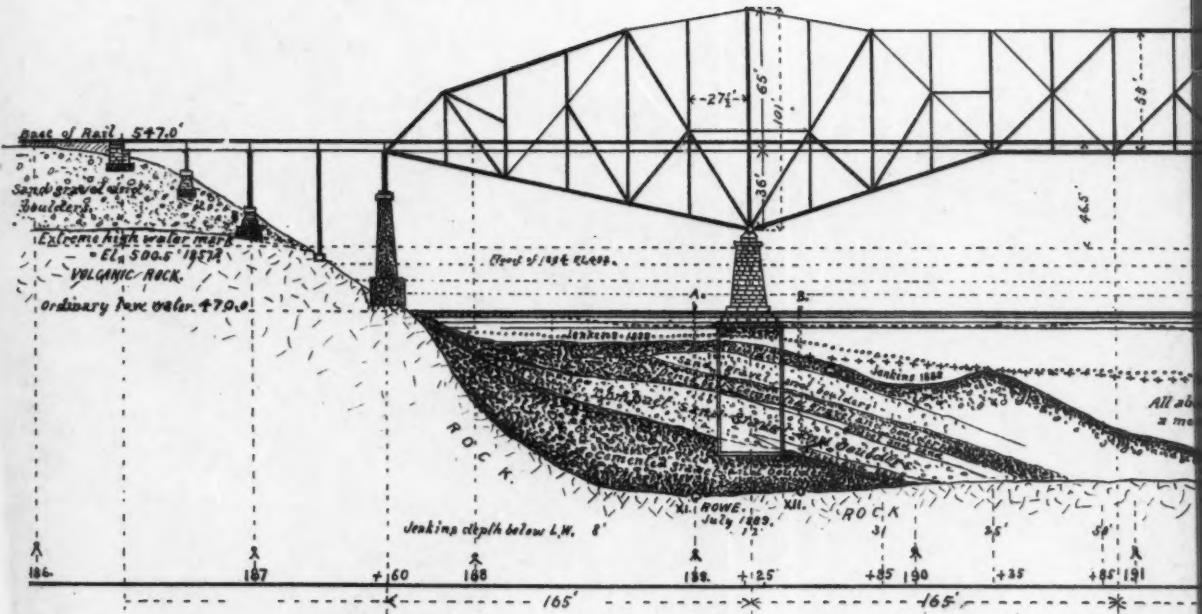


PLATE CXVI.  
TRANS.AM.SOC.CIV.ENGRS.  
VOL.XXV. N° 518  
RED ROCK CANTILEVER BRIDGE.

## Atlantic & Pacific Railroad.

## *RED-ROCK BRIDGE*

(Cantilever.)

Topeka Kansas, Feby 28<sup>th</sup> 1891.

Sam M Rose  
chf Engr.

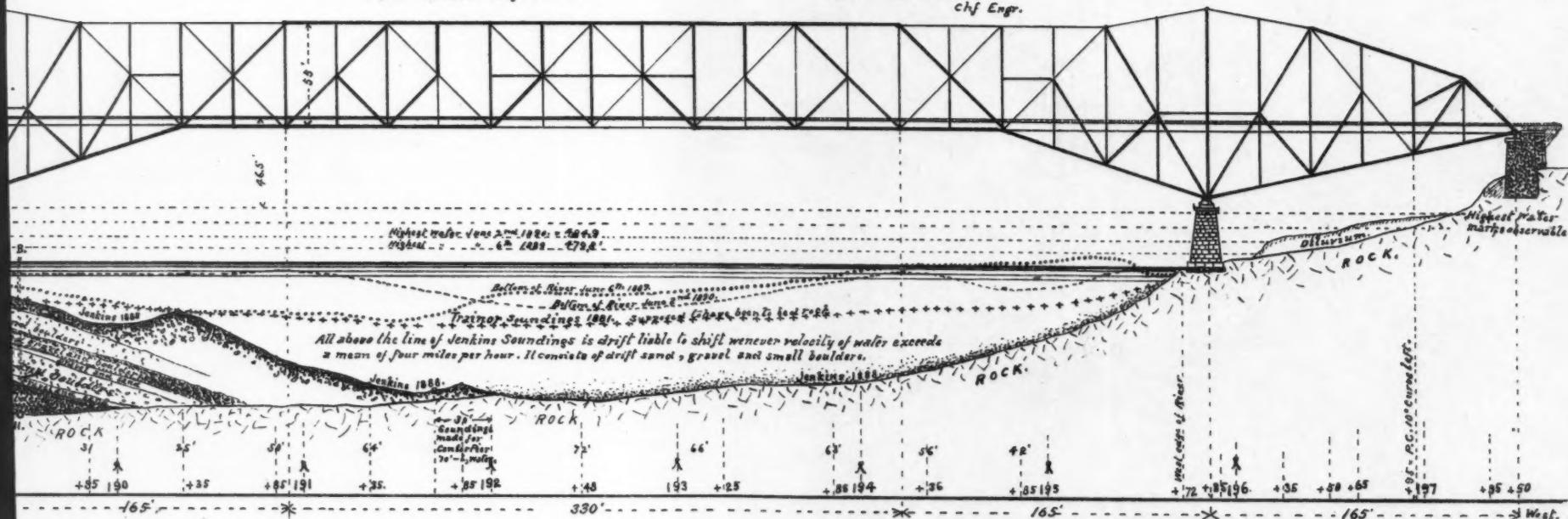
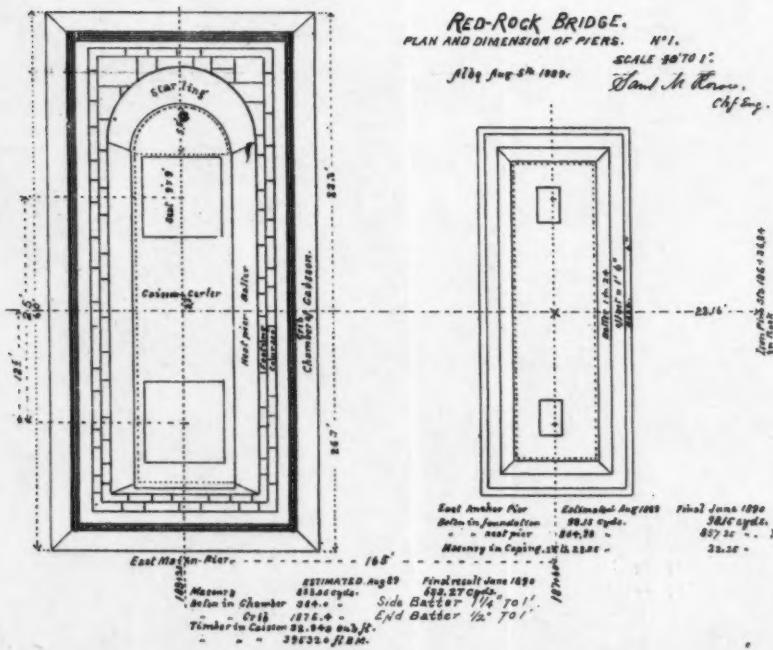
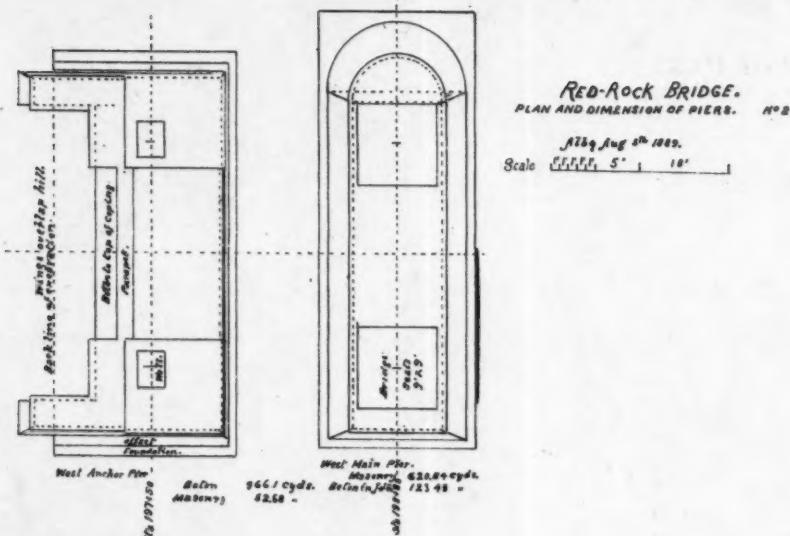
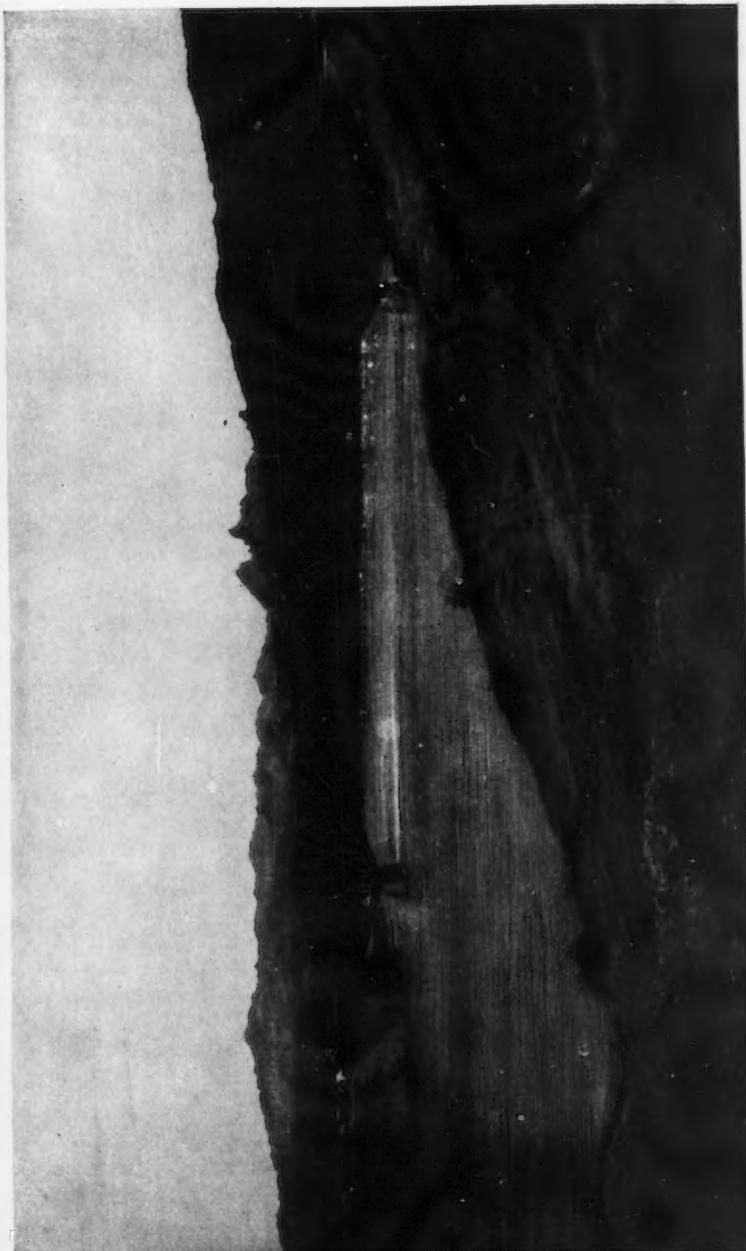


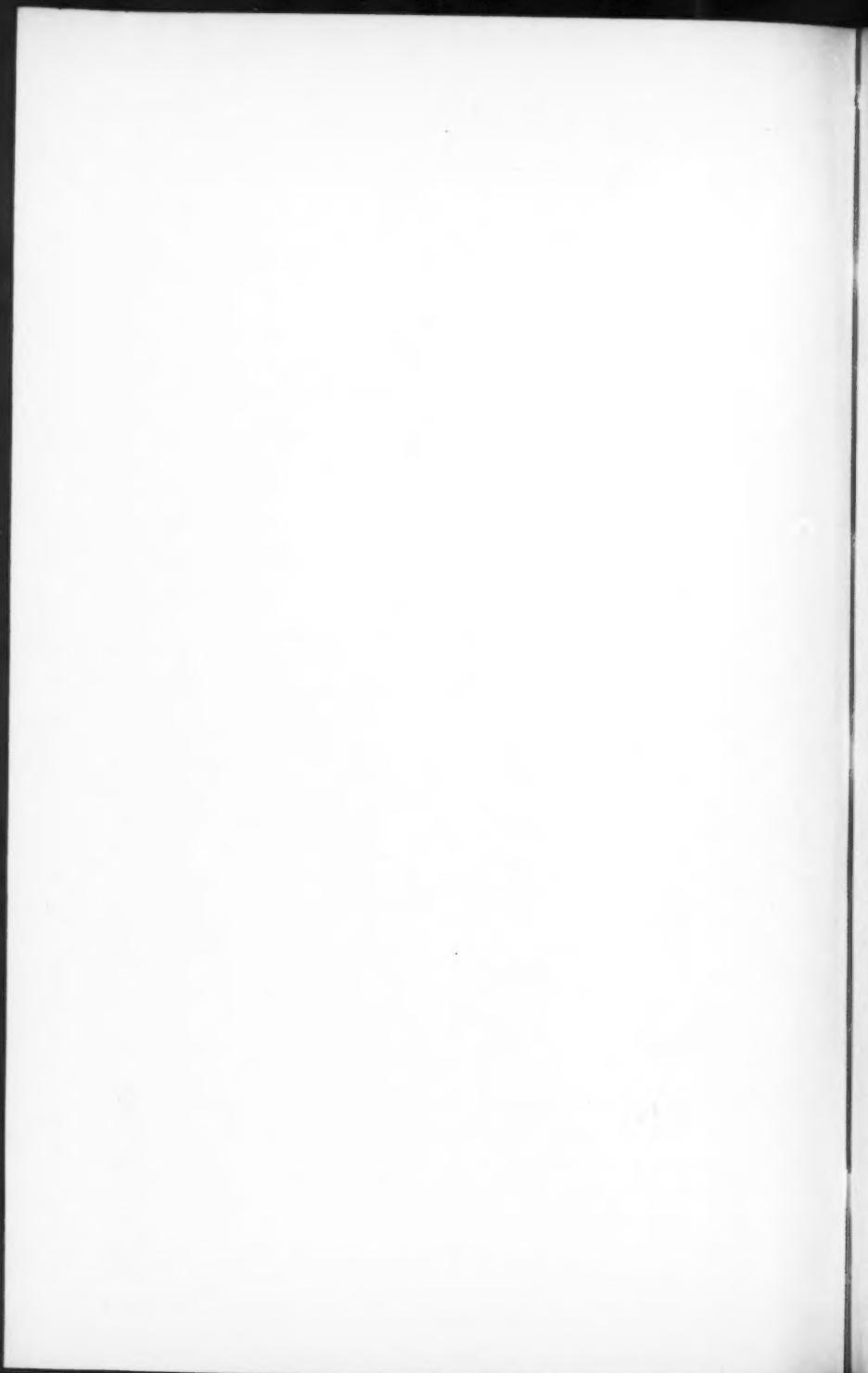
PLATE CXXI.  
TRANS. AM. SOC. CIV. ENGR'S.  
VOL. XXV. NO 518.  
RED ROCK CANTILEVER BRIDGE.





[PLATE CXXII.  
TRANS. AM. SOC. C. E.  
VOL. XXV, No. 518.  
RED ROCK CANTILEVER BRIDGE.]





## RED ROCK BRIDGE.

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### GENERAL SPECIFICATIONS AND PROPORTIONS.

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By S. W. ROBINSON, M. Am. Soc. C. E.

Among the questions of importance concerning the Red Rock Cantilever Bridge, consideration of which was shared by me, may be mentioned the general specifications for the superstructure; the form of structure; its economic general proportions; and the results of test of materials, and members.

SPECIFICATIONS.—Those adopted were those of the Phoenix Bridge Company for 1888, amended so as to provide for a heavier engine load, viz.: two of the heaviest engines of the Atlantic and Pacific Railroad, each with a forward truck having two pairs of wheels, and weighing 22 000 pounds; three pairs of drivers, weighing 92 000 pounds on 11 feet 8 inches; and a tender weighing 74 000 pounds; or a total of 188 000 pounds on 49 $\frac{1}{2}$  feet. Also to provide for a full setting of the train brakes on the viaduct approach; to reduce the maximum unit strains allowed by about 6 per cent.; and for making the pins of steel of 70 000 pounds ultimate tensile strength, with duly proportioned connections. The amended specifications increased the weight of the floor system about 20 per cent. and trusses about 7 per cent. over what is to be regarded as good present practice; thus assuring ample capacity of the bridge for unusually heavy engines and loads, and the probable increased weight of rolling stock and loads of years to come.

TYPE OF STRUCTURE.—According to the most reliable information at hand as to the river bed, when the general form of superstructure was adopted, the cantilever was agreed upon as the most suitable for this location. As expedition was too urgent to allow time for multiplied trial strain sheets seeking the most economical proportions, and to accord with relatively short anchor arms, an unusually long center suspended truss was accepted. When the most economic proportions were

settled, it was too late to make changes, had it been desirable, but the half span length given the suspended truss, was found, everything considered, about the best.

ECONOMIC LENGTH OF CENTER SUSPENDED TRUSS.—Reason for an excessive length of this center truss was found, in the fact that the structure was to be chiefly of steel. Quite extended calculations on both a center suspended truss of half the main span length and also a third of the span length, taking account of all loads, dead, live and wind, furnished grounds for the following statements, viz.:

1st. For maximum unit strains for steel, by the formula  $s \left( 1 + \frac{\text{min.}}{\text{max.}} \right)$  of the specifications adopted, the half span length of center was more economical than the third span length. 2d. For maximum unit strains for steel by formula (s) there was no difference in economy. 3d. For maximum unit strains for iron by formula  $s \left( 1 + \frac{\text{min.}}{\text{max.}} \right)$  there was no difference in economy for the two lengths. 4th. For maximum unit strains for iron by formula (s) economy favors the shorter center. 5th. For iron, the most economic length of the center suspended truss is shorter than for steel. 6th. The lower the maximum unit strain allowed in a given case, the shorter will be the most economical length of center truss as compared with the main span.

In a subsequent examination of the question it was found that by the formula :

Allowed unit stress =  $s \left( 1 + \frac{\text{min. total stress}}{\text{max. total stress}} \right)$  in the case of a center suspended span of  $\frac{1}{2}$  the length of the main span, there was found a saving of  $1\frac{1}{6}$  per cent. of steel, as compared with the case of a one-half span length of center truss. For iron it is probable that  $\frac{1}{6}$  would be found a more economic ratio than  $\frac{1}{2}$ . The main span is 660 feet in length. The disadvantage of 1.6 per cent. found above, or about \$2,000, on the structure, is probably fully met in the greater lateral stability of the bridge as due to the adopted half-span length of center truss.

LATERAL STABILITY.—This question of lateral stability became an important one when the adopted width of about  $\frac{1}{7}$  the span was assailed by those of good judgment and standing, and also in view of the fact that work on the bridge was in progress. Accordingly answers were soon given proving that the  $\frac{1}{7}$  in this case was as good as the  $\frac{1}{5}$  of span

for simple bridges, as adopted by good designers. Since then the question has been further investigated in a general way, and from two different points of view. The results of the investigation are here given, and establish the 25-foot width adopted as ample.

*First.*—For a general formula the elasticity of the laterals and of the chords was considered. In this, it is readily seen that if the laterals in the cantilever and simple bridge are at the same angular positions in plan, their elongations and compressions for maximum strains will be the same in the one as the other, so these can be disregarded.

Next, as to the chords, considering the lateral strain as due to uniform load, the strains in the chords will be similar in distribution to those due to maximum vertical loads, and the unit strain nearly uniform; so that the chords elongate or shorten very nearly a constant amount throughout, for a given case. Hence the lateral deflections due to strains in the chords would be those due to the shifting of parts in plan of the bridge from the straight line, to circle arcs of common radius. This furnishes a rational basis for calculating deflections.

Next, as to the end conditions, the fairest assumption for the cantilever arms is that they are fixed to their piers, and that the suspended truss is jointed to them. Also the simple bridge compared with, should be regarded as fixed at one end to its pier or abutment, while the other end is in effect jointed to the pier because one end usually is on expansion rollers.

Then, if  $l$  be the length of span, of either a cantilever or simple bridge,  $nl$  the length of the center suspended truss of the cantilever,  $b$  the width of the cantilever center to center, and  $b^1$  that of the simple bridge, we find for the deflection of the end of the cantilever arm—

$$\frac{s_1 l^2}{E b} \left( \frac{1-n}{2} \right)^2,$$

and for the center truss, its middle point compared with its ends,

$$\frac{s_2 l^2}{E b} \left( \frac{n}{2} \right)^2$$

all parts of the cantilever being supposed deflected into horizontal circle arcs of the same curvature, as above explained.

Also of the simple bridge of length  $l$  and width  $b^1$ ,

$$\frac{2 s_3 l^2}{E b^1 (2 + \sqrt{2})},$$

which last is to be placed equal to the sum of the first two expressions,

that is, the total deflection of the cantilever bridge is to be the same as that of the simple bridge, in which  $s_1$  and  $s_2$  are unit stresses, due to lateral loading, and  $E$  the co-efficient of elasticity, taken the same in the simple truss as for the cantilever trusses.

The last expression supposes the simple bridge, fixed at one end and supported at the other, and the curve of deflection to be composed of two circle arcs in common tangency at a point of contraflexure at a distance from the fixed end  $= \frac{l}{2 + \sqrt{2}}$ , and which supposes the supported end to be in the direct line of the tangent to the curve at the fixed end. This relation is readily proved by trigonometry.

Hence, writing the equation we get—

$$\frac{b}{b^1} = 1.45 \left( 1 - 2n + 2n^2 \right) \frac{s_1}{s_2}.$$

If  $s_1$  differs from  $s_2$ , the latter is probably greatest because of the greater depth of the cantilever arm at the pier, while its breadth is the same throughout the bridge.

But taking  $s_1 = s_2$  we obtain—

For $n = \frac{1}{2}$ .....	$\frac{b}{b^1} = .725$	the minimum.
" $\frac{1}{4}$ .....	" .806	
" .4 .....	" .754	
" .81 .....	" 1.000	

If the simple bridge with a span of 660 feet be assumed 36 feet wide, or about  $\frac{1}{15}$  the span, this cantilever with a 330 feet suspended truss will have equal lateral stability when 26 feet wide for  $n = \frac{1}{2}$ , and for  $n = .4$  when it is 27 feet wide. The bridge was built 25 feet wide, center to center, and some 29 feet wide at the out to out of the heavy chords of the cantilever arms, a width which is equivalent to about 28 feet for purposes of lateral stability or stiffness.

2d. A formula may be obtained on the supposition that the bridges in deflection will follow the same law as solid beams. Then the moment of inertia of the sections will vary as the cube of the breadth of bridge.

Hence, from usual formulas for deflection, with the end conditions as before—

$$.0054 \frac{wl^4}{Eb^{13}} = \frac{wl^4}{3 Eb^3} \frac{n}{2} \left( \frac{1-n}{2} \right)^3 + \frac{wl^4}{8 Eb^3} \left( \frac{n}{2} \right)^4 + \frac{5 wl^4}{384 Eb^3} \left( \frac{2}{n} \right)^4;$$

or, 
$$\frac{b^3}{b^{13}} = 3.86 \left( n(l-n)^3 + \frac{n^4}{2.42} \right).$$

The minimum value of this occurs at,

$$n = 0.53 \text{ when } \frac{b}{b^1} = .695 \text{ the minimum.}$$

and for $n = .5$	"	.698
" .4	"	.72
" .3	"	.74

The two formulas agree essentially in results, and give the width of the bridge at practically the same figure as that adopted in the construction. These formulas indicate a slightly wider cantilever than those obtained at the time of the first investigation, owing to the fact that there the simple bridge placed in comparison was regarded as supported or jointed at both ends, while evidently it should be jointed only at one end and fixed at the other as herein considered.

The width, 25 feet, compared with the span is as 1 to 26.4, which compares favorably with several notable bridges regardless of type, viz.:

The Ohio River Bridge, Cincinnati, O.....	1	in 26.
Britannia Tubular Bridge.....	1	" 38.3
Niagara Suspension Bridge.....	1	" 34.
Victoria Tubular Bridge.....	1	" 21.
Freiburg Wire Bridge.....	1	" 41.
Niagara Wire Bridge.....	1	" 33.
Union Iron Suspension Bridge.....	1	" 25.

In the light of these precedents as well as from the results of analysis, there can exist no question as to the judicious selection of the width of the Red Rock cantilever.

The question of widening the cantilever at the piers without changing the center was carefully considered and advised adversely for two reasons: first, on account of the complication of shop work; and, second, from the outward thrusts at the bridge seats tending to split the pier as the cross-member at the seats expand and contract by heat and cold.

SECTION OF SUSPENSION AND ANCHOR BARS.—One question which appears to have been overlooked heretofore in cantilever bridges, is that of the increment of stress in the main anchor and suspension links, due to the expansion and contraction from changes of temperature of the bridge, and consequent unavoidable springing of the links, or else slipping of them around on their pins. Evidently within certain limits the links will spring, beyond which the friction being overcome, they will slip on their pins. At first, in new bridges, the co-efficient of friction will be between 0.2 and 0.3, when later, by abrasion or by corrosion, there

will be a sticking fast of link and pin. Entire safety of the structure demands that the links be so proportioned as to spring without slipping while the strains remain within the allowed limits. Granting that the pin will never stick fast but that the co-efficient of friction, when the construction oil is exhausted, is the same as for a locomotive wheel on the rail, viz.: 0.3, the stress in the neck of the link to slip it on its pin is something surprising. Thus, equating the moment of friction with the moment of stress in the neck of a link we get—

$$tbdfr = \frac{sbd^2}{6};$$

whence,

$$\frac{s}{t} = \frac{6fr}{d}$$

in which  $t$  = the unit tension load on a link,  $s$  = the unit stress in the neck of a link due to the bending moment caused by friction,  $b$  = thickness,  $d$  = breadth of neck of link, and  $r$  = the radius of pin.

If, for example, a pin be 8 inches in diameter, the link 2 x 8 inches and the co-efficient of friction =  $f = 0.3$ .

$$\frac{s}{t} = 0.9,$$

that is, where the breadth of a link is the same as that of the pin on which it turns, the added stress in the link due to pin friction is  $\frac{1}{10}$  that due to the suspended load; from which it is plain that any suspension or tie link intended to swing loaded on a suitable pin must be largely increased in section above that of a link subject to ordinary conditions.

But the anchor and suspension links in this Red Rock bridge were so long as not to approach the point of slipping on the pin, the whole expansion of the trusses being taken care of by the springing of the links. The superadded stress due to spring was calculated and provided for in proportioning these links. A formula for this was obtained upon the supposition that the link is fixed at both ends to its pins, and with a point of contraflexure at the middle, the half link being thus conditioned like a beam fixed at one end and free at the other. Then the well-known formulas for flexural stress and elastic deflection give

$$Pl = \frac{sbd^2}{6} = \frac{3 EI \Delta}{l^2} = \frac{Ebd^3 \Delta}{4 l^2};$$

whence the flexural stress,  $s = \frac{3 \Delta d}{2 l^2}.$

Taking  $E$  at 25 000 000,  $d$  at 8 inches,  $l$  at 25 feet, and  $\Delta$  the deflection at 0.6 inches as for the suspension links in this bridge:

$$s = 2000 \text{ pounds per square inch},$$

a value less than that required to turn the link on the pin, so that the link will not turn; but the 2 000 pounds is to be included in the maximum allowed stress, and hence the links must be some 12 to 16 per cent. thicker than if this unavoidable flexural stress is neglected. The anchor links were likewise provided for, besides requiring the lengthening of those at one anchorage beyond the preferred length, to bring the flexural strains within suitable limits.

**QUALITY OF MATERIALS AND MEMBERS.**—The requirements of the specifications respecting qualities were physical and not chemical, and the reports of the tests gave evidence of remarkable uniformity of test values.

The only matter worthy of note here was, 1st, the fact that two out of the eight full sized eye-bars selected for test gave much lower ultimate resistances than was expected, judging from the "heat tests." The requirements of the specifications, and results of the two full sized eye-bars in question, are grouped in the following table :

NAME OF SPECIMEN.	Length, feet, of specimen.	Section of specimen in inches.	Ultimate unit tension.	Least ultimate unit tension.	Least elastic resistance limit, $\equiv \frac{1}{2}$ ultimate.	Percent of elongation in 10 least diameters.	Per cent. reduction of section.
Required by Specifications .....	....	....	62 500	58 000	$\frac{1}{2}$ ultimate.	20.5	41.
First eye-bar .....	28.5	2 x 8 inches.	....	48 345	27 570	31.6	53.1
Heat test of same .....	....	" "	....	61 060	36 460	27.	61.
Second eye-bar .....	28.5	2 $\frac{1}{2}$ x 8 "	....	50 120	28 915	33.7	48.
Heat test of the same ...	....	" "	....	63 550	37 590	26.5	62.4

For these finished eye-bars, all the test figures show higher values than those required by the specifications except for the ultimate resistance, and the fractured section was noted as silky. The heat tests show no exceptions to the specifications. Comparing the latter with the former, there is seen a general drop in values, indicating a depreciation in quality. But this very largely is to be accounted for in the much greater section under test in the eye-bar than in the three-quarter-

inch heat test specimen, lower values for large sections being well known to be usual.

Though the low ultimate tension in the finished bars was to be regretted, yet the ductility and fibrous structure were so thoroughly established as to assure confidence in the final acceptance of the bars. In another eye-bar a flaw in the head was discovered by the inspector, when the bar was set aside for test purposes. This bar broke in the flaw at about 77 per cent. of the value obtained in the neck after reheading and annealing. The fracture in the head had a crystalline appearance devoid of fiber, as if the steel were slightly burned while forging the head, the same probably being one cause of the flaw. The neck of the bar when it was broken gave an ultimate unit resistance of 56,360 pounds, which, as in case of the other two, is also somewhat lower than the specifications call for.

With some samples of pin steel there was a similar experience, partly explained, however, in the fact that some specimens were taken from a diametrical position in the bars.

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## RED ROCK BRIDGE.

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### SUPERSTRUCTURE.

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By HENRY H. QUIMBY, M. Am. Soc. C. E.

The location of the piers in determining the length of the river span and the anchor arms, gave proportions unusual, and for economy, undesirable in such structures. But the relative shortness of the anchor arm is of manifest advantage in securing uniformity in the character of the strains in all the members of that portion of the truss. The dead weight of the structure between the piers is sufficient to overbalance the combined live and dead load on the anchor arm, and by consequence, reverse strains cannot occur in any member. This condition would permit the use of tension members in the upper chord of the anchor as well as cantilever arm and the end braces, but it was deemed advisable to make each anchor arm a complete truss competent to support itself and

the heavy traveler in the event of disaster to the falsework. Therefore the upper chord panels affected, and the end braces, were built of 24-inch latticed chords and 8-inch eye-bars in combination. The trusses were at first designed to have the top chord level over the piers and river span, but the change, increasing the depth 10 feet over the piers, relieved the long inclined posts of a considerable portion of their load and effected a better distribution of the pressure on the pin and shoe, besides giving a more pleasing outline to the bridge (Plate CXXXIII).

Facility of erection had to be kept constantly in mind in designing the arrangement and connection of members, and in some cases the strains produced by the traveler determined the section. The members in the top and bottom panels between the center span and lever arm, have no other duty than that of forming the chords of the lateral trusses. In the erection, however, they were subject to tensile and compressive strains about equal to those in the corresponding panels of the great Forth Bridge. The computations were based on the following loads:

*Lieu.*—Two engines each weighing, with tender, 94 tons, concentrating 46 tons on a wheel base of 11 feet 9 inches, and followed by 3 000 pounds per foot.

*Dead.*—Track, 450 pounds per linear foot; floor system, 450 pounds per linear foot; trusses of anchor and lever arms, varying from 3 200 to 4 100 pounds per foot; trusses of center span, 1 550 pounds per foot.

*Wind.*—Thirty pounds per square foot on exposed surface of a train and both trusses.

The allowed unit stresses as prescribed by the Consulting Engineer, Prof. S. W. Robinson, were in general somewhat lower than those of the Phoenix Bridge Company's standard specifications, under which in all other respects the bridge was designed, and were for—

Steel eye-bars and verticals 9 200.

Diagonals and chords 9 400  $\left(1 + \frac{\text{min.}}{\text{max.}}\right)$  whenever this quantity exceeds 12 300.

Plate hangers 8 000 pounds.

$$\text{Compression members, flat ends, } P = \frac{10\,000}{1 + \frac{l^2}{40\,000 r^2}}$$

$$\text{Compression members, pin ends, } P = \frac{9\,730}{1 + \frac{l^2}{30\,000 r^2}}$$

Pins and rivets,	9 000 pounds for combined live and dead loads.	
shearing.....	12 000	" for wind and erection.
	11 000	" for wind and live in combination.
bearing .....	14 500	" for combined live and dead.
	20 000	" for wind or erection.
	18 000	" for wind and live load in combination.
bending on ex-	20 000	" for combined live and dead loads.
treme fibers....	26 000	" for wind or erection.
	24 000	" for combined wind and live.

For field connections the number of rivets to be increased 25 per cent.

The material used throughout, except in the floor, adjustable rods, upper struts, and lacing of chords, is open hearth steel, for which the requirements were:

Ultimate—for tension steel—between 58 500 and 66 500.  
for compression—between 64 000 and 72 000.

Elastic limit—not less than one-half the ultimate tensile strength.

Elongation in 8 inches—not less than  $\frac{1\,200\,000}{\text{ult.}}$  per cent.

Reduction of area—not less than  $\frac{2\,400\,000}{\text{ult.}}$  " "

The erection stresses were due to the weight of the overhanging structure with the track and the traveler, the latter weighing 240 000 pounds, which, with the weight of pieces being raised in position, was all concentrated on the front wheel. The resulting strains in the top panel of the lever arm (Plate CXXIV) were twice as great as those to be provided for in the finished structure, and these panels were consequently reinforced during erection by the addition of a double line of 24-inch square iron bars on the midline of the truss. Temporary diagonals, made of adjustable rods and stirrups, were employed to support the middle of the end post of the center span while the traveler was at that point. All the joints of the top chords of the center span were proportioned for the tension during erection and were made forward of the pins. The bottom chord panels, subject to compression in erection, were made of 24-inch built channels, latticed.

Each truss is anchored (Plate CXXV) with four  $8 \times 1\frac{9}{16}$ -inch eye-bars secured by an 8-inch pin to a 24-inch bolster, 8 feet long, which carries one end of two 50-inch plate girders extending the whole length of the piers.

On these girders rest fifteen 15-inch rolled beams embedded in the mass of concrete. At the west end the bars are connected to the truss heel by a short, stiff member which also carries the end floor beam. At the east end sufficient anchor weight was secured in the pier without carrying it up to the truss, and for economy the concreting was stopped 14 feet 6 inches below the heel pin, the anchor bars in the intervening space being made of 24-inch built channels braced transversely by heavy latticed diagonal struts, designed to transmit to the top of the pier the horizontal wind thrust from the trusses. The connection to the pier top is by sixteen bolts  $1\frac{1}{8}$  inches diameter in slotted holes, permitting no lateral motion, but a slight longitudinal as well as vertical movement due to thermal changes and deflection. At the west end the floor beam, while not resting on the pier, is bolted to it with similar liberty of movement.

The pedestals on the main piers are each 7 feet 3-inches square, and consist of eight ribs in a transverse direction united by heavy top and bottom plates. The truss shoes resting on the pedestals have four longitudinal ribs with a total thickness of 12 inches, which forms the bearing of the 15-inch main pin on which abut the five principal compression members.

The center span is suspended by four  $8 \times 2\frac{5}{8}$ -inch eye-bars from the upper end of the lever arm, and swings freely in oblong pin-holes at both ends. The floor beams throughout are riveted to the vertical members. In the anchor and cantilever arms the stringers, being near the neutral axis of the truss, are riveted to the webs of the floor beams; but in the center span, where they are in the plane of the lower chord, they are carried by brackets on the floor beam, and provided with slotted holes to permit the slight longitudinal motion caused by the stretch of the lower chord consequent upon the deflection of the trusses.

The transverse and horizontal bracing (Plate CXXVI) is of latticed struts and upset rods, with clevis attachment, except between the main inclined posts and along the lower chord of the anchor and cantilever arms, where the diagonals also are lattice struts, in the latter case 30 inches deep. The main inclined posts and all the verticals in the arms have transverse bracing below the floor. The verticals over the piers are 101 feet long, with one splice, and the inclined ones adjacent, 107 feet, also made in two lengths. Those in the cantilever arm, having an accumulation of

top wind load to transmit to the pier, are stiffened against bending at the point of attachment of the transverse bracing. The pins generally are 8 inches in diameter, and the shoe pins (Plate CXXVII) are 15 inches in diameter, 5 feet long, and weigh 3 000 pounds each.

The lower lateral system of the center span ends, and that of the cantilever arm, begins at the point where expansion takes place, the shear being transmitted from one system to the other by lateral contact, one member sliding within the other. Although all the wind load from the top chord of the center span is assumed to pass down the end braces into the lower system, as (being its most direct road to the pier) the upper system of rods is carried through the expansion panels and a similar device employed to transmit shear.

The track rails are steel, 61 pounds per yard, with ordinary four-hole angle splices and spiked with  $\frac{1}{6}$ -inch square x  $5\frac{1}{2}$ -inch spikes. The cross-ties are white oak 8 x 12 inches x 10 feet, set on edge,  $13\frac{1}{2}$  inches center to center, and notched three-quarters of an inch over stringers. On each floor-beam is a tie laid flat. Every third tie is 16 feet long, carrying two walks, which are each made of two 3 x 12-inch planks 9 inches apart. The outer guards are 6 x 6 inches Southern pine, notched  $1\frac{1}{2}$  inches over ties, spliced with an 8-inch scarf, and bolted through every third tie to each stringer by a  $\frac{1}{2}$ -inch bolt passing through the stringer flange. These guards are directly over the stringers, 8 feet center to center. The inner guards, which have not yet been put on, are designed to be 4 x 4 x  $\frac{1}{2}$ -inch angle iron set 8 inches from the rail head and bolted with  $\frac{1}{2}$ -inch lag screws to every tie.

The calculation of the lengths of members was made with a view of having them in their normal position when fully loaded. In the center span the trusses were cambered 3 inches by increasing the top chord panels and the diagonals proportionately, adding to compression members and deducting from tension members the compression and stretch due to the dead load only, using a modulus of elasticity of 26 000 000 pounds. For the arms the lengths were figured with all the pins at their normal elevation, but the allowance for compression and stretch covered the whole load—live and dead. The effect on the arms was to shorten the whole top chord and lengthen the whole bottom chord, consequently raising the end of the cantilever arm. The allowance in the long inclined posts increased their length  $\frac{1}{6}$  of an inch, and the hangers depending from their tops were shortened one-quarter of an inch. The

members at this point form a rigid quadrilateral, tied diagonally, and with the allowances acting in opposition, the connecting of those in the lever arms necessitated the opening of the first joint in the lower chord sufficiently to raise all the members meeting at the lower end of the shortened hanger, until the connecting pin could be driven. The joint closed gradually as the traveler moved forward, and when the end of the cantilever arm was reached, enough strain had been produced to compress and stretch the members sufficiently to close the joint completely.

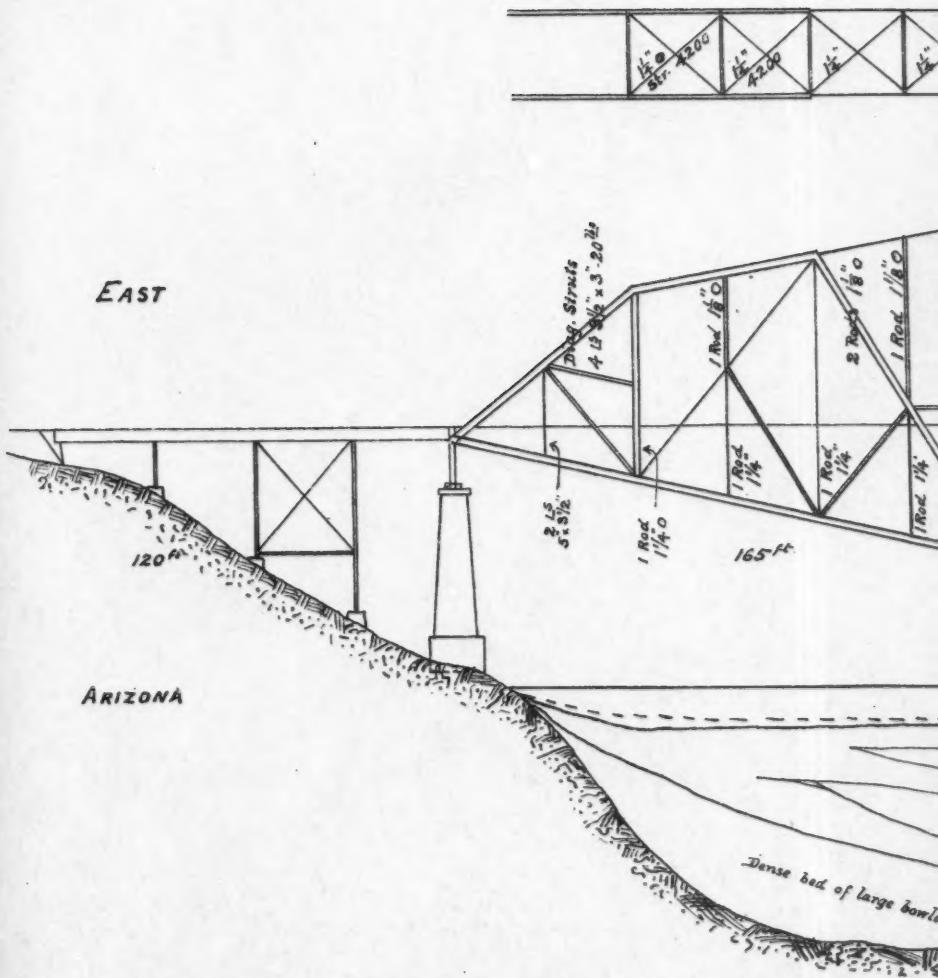
Some uncertainty was felt as to the accuracy of the setting of the pedestals on the piers as affecting the length of span. They were the starting points of the erection on their respective sides, and it would be impossible to shift them after the projecting arms were built. The great distance that the bridge is from the works, requiring four weeks for the transport of freight, made it impracticable to leave a panel at the center of the span to be made after the two halves had almost closed the gap and the length of the closing panel measured. So the shop work was completed and provision made in the adjustment panel for a possible error in length of span. The stringers in these panels (which are the ones immediately adjoining the center span), are carried in deep pockets on the floor beams, and have oblong holes at each of their ends. The pin-holes at one end of the chords in these panels were made with 9-inch play. The adjustment was by wedges (Plate CXXVII) sliding between planed surfaces, the ordinary device of working the wedges between rollers being impracticable because of the great pressure. The wedges were built of seven-eighths-inch vertical plate ribs and seven-eighths-inch plate faces united by angle iron, with their surfaces polished. The slope of each face was 1 in 12, making the wedge 1 in 6. The angle of rest with lubricated surfaces being about 1 in 8, as shown by recorded experiments, the wedges should squeeze out under pressure, and to prevent this and to move them in or out, as occasion might require, 3-inch screws were passed through and secured to them, with nuts bearing on an iron frame underneath and wooden blocking above. The lower wedges bore against the projecting beveled end of the fixed chord section, and a loose beveled box pinned to the movable section, the latter being pivoted, adjusted itself to the slope of the wedge. The upper wedges each worked between two pivoted boxes, so arranged that the operation of driving in the wedge would shorten the panel and th-

whole projecting top chord, at the same time raising the center. The faces of the boxes were seven-eighths-inch plates, supported by horizontal ribs. In erection, the wedges were all set in far enough to avoid the necessity of driving afterward, although screws were provided to make this possible if the occasion should arise. The lower wedges were entered to increase the panel length 3 inches, thereby raising the center point 9 inches above the normal level, at the same time permitting a connection at the center if the span should prove to be as much as 6 inches too long. The upper wedges, similarly set, gave additional elevation at the center, and allowed for a possible shortage in the length of span. The dead pressure on each wedge was 656 000 pounds, or 910 pounds per square inch of bearing on the lower wedges, which are 30 inches wide, and 1 370 pounds per square inch on the upper wedges, 20 inches wide. As all the wind load on the projecting half span was assumed to pass through the lower wedges (that from the top chord passing down the end braces), the pressure per square inch on the latter might be increased to 1 200 pounds.

As the surfaces were to remain in close contact for several weeks in a hot climate, with the sun's heat tending to melt out the grease, and with no possibility of introducing any after setting, a mixture of plumbago, with just enough tallow to hold it in mass, was spread thickly over the sliding surfaces and proved effective in preventing abrasion.

The anchor arms were erected on falsework (Plate CXX). The pedestals and shoes were first set and then the lower chords and struts were placed in position by a small traveler. All the remaining material was handled by the large traveler, which carried three hoisting engines and erected the trusses and floor from its overhanging front, and had two booms at the back to erect the bracing behind it. Eight of the track stringers were reinforced with cover plates and fitted with rails to form a track for the traveler. These rested on top of the floor beams, which were carried by the stiffened hangers, those of the anchor arm acting as posts. The permanent track was laid as the traveler advanced, and was used for running out the material which was passed between the traveler legs to the front.

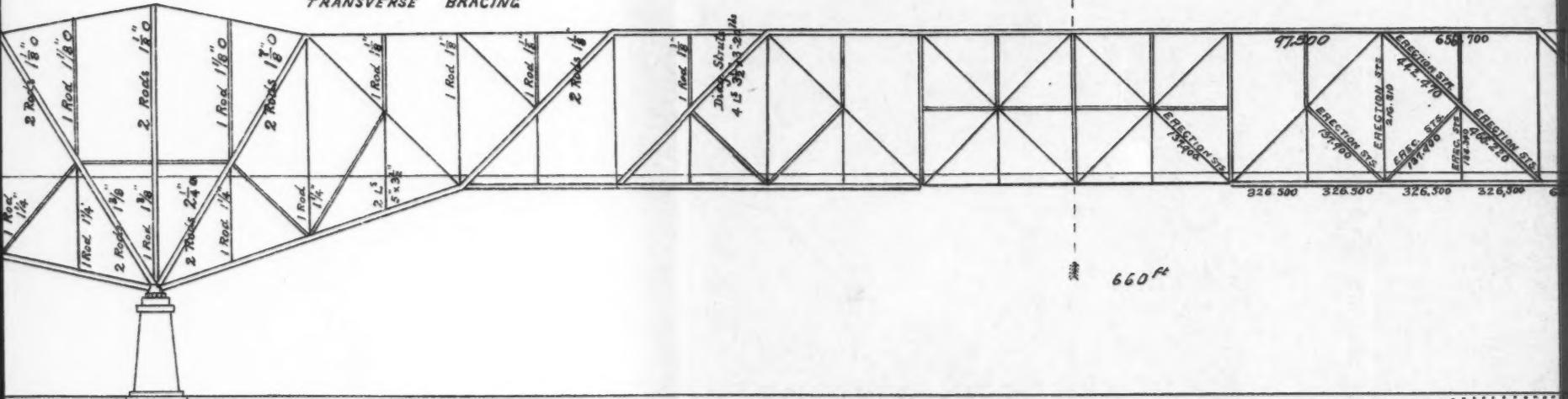
The most interesting part of the erection was the final connection at the center. For six weeks the west half of the structure had been standing with 330 feet projecting over the river, and when the east half reached it, the pin holes in the lower chord eye-bars lapped  $5\frac{1}{4}$  inches; showing, as the wedges had pushed each half 3 inches out, that the span



## *UPPER LATERALS*



### **TRANSVERSE BRACING**

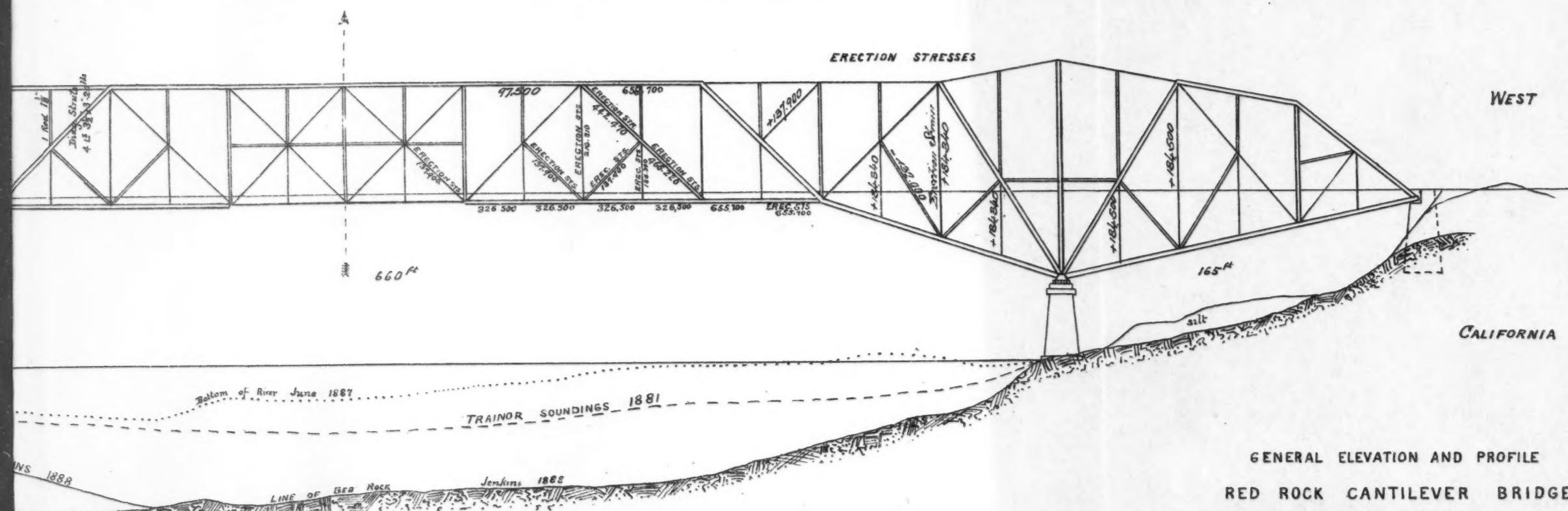
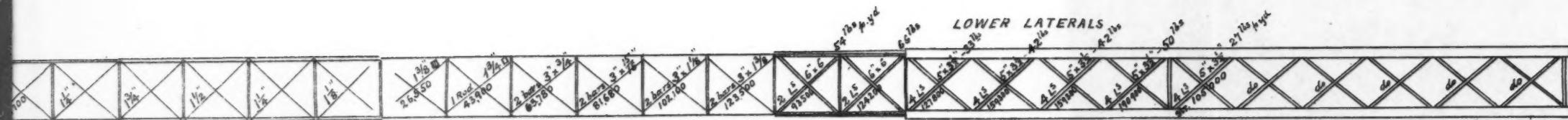


Bottom of River June 1887

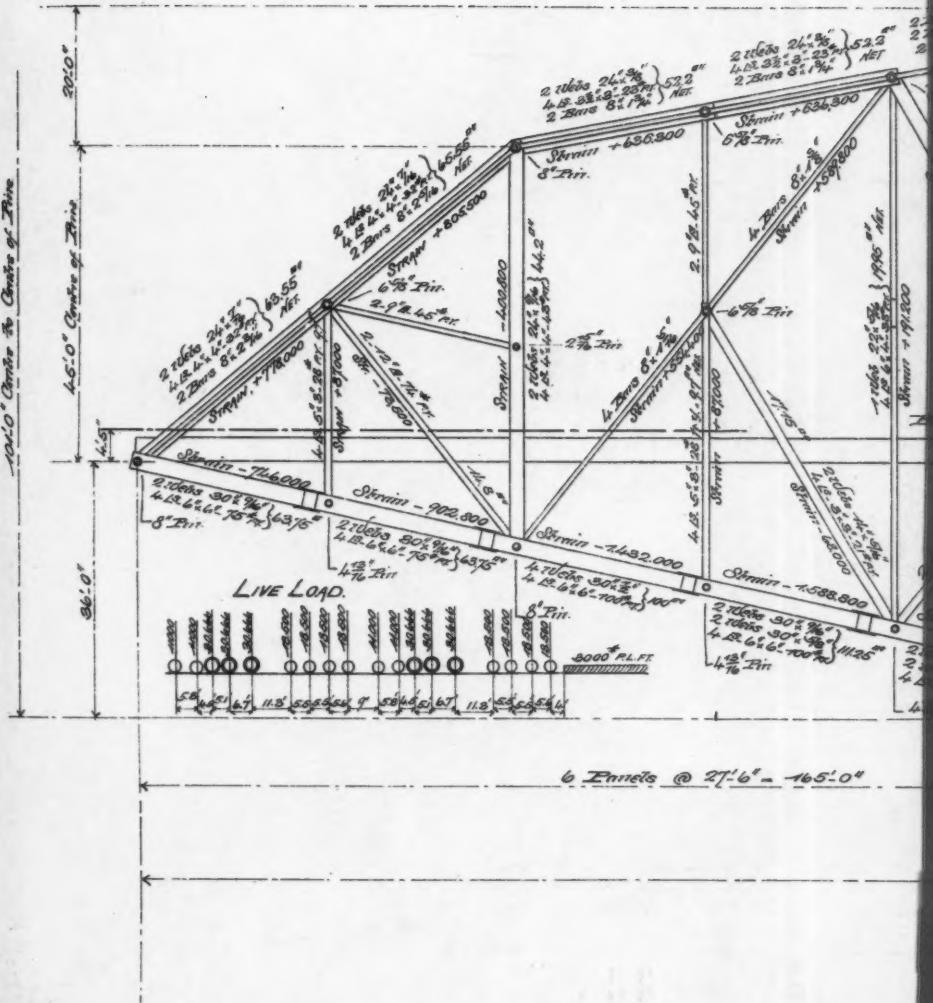
TRAINOR SOUNDINGS 1881

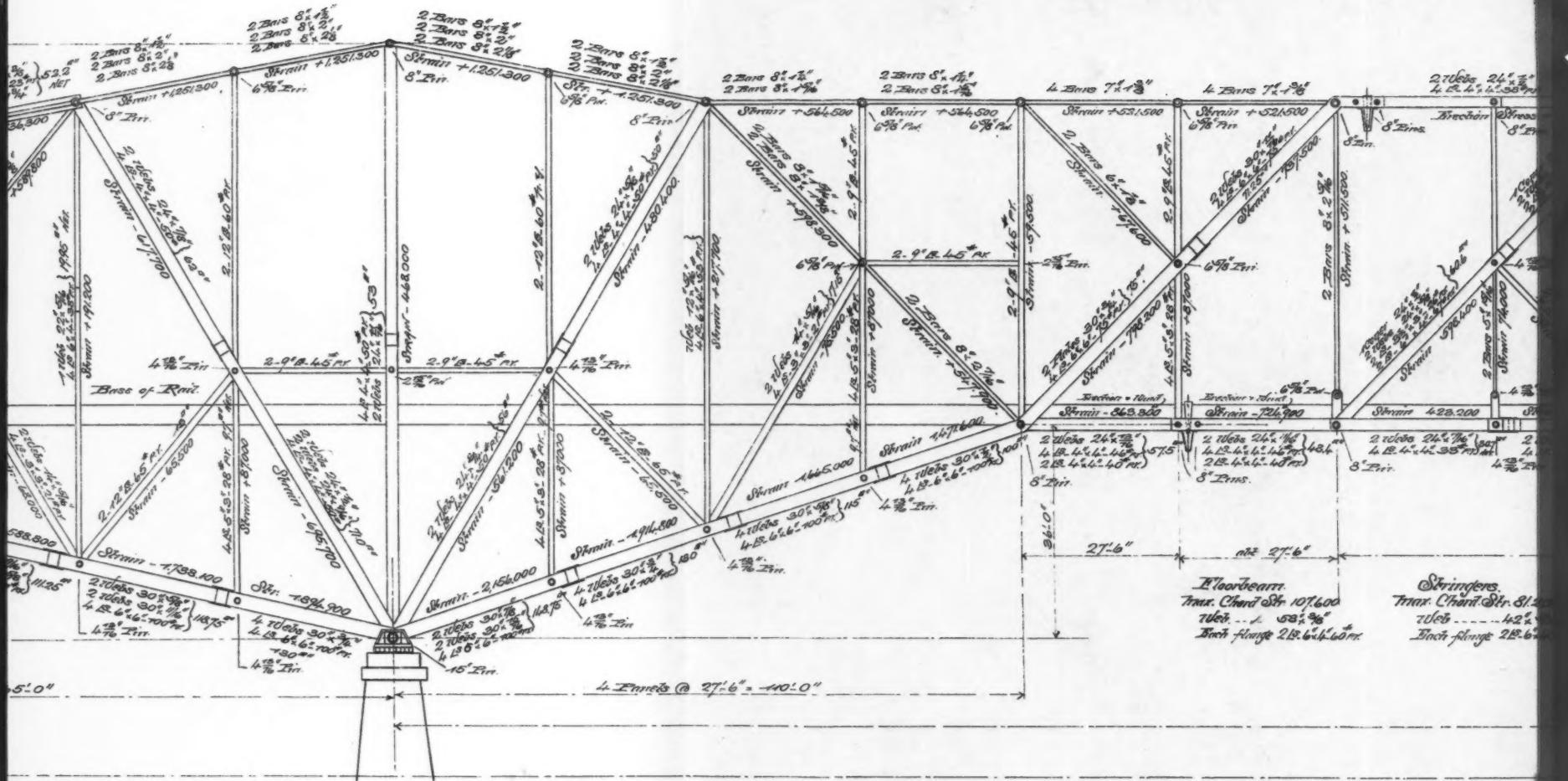
A detailed cross-section diagram of a river bed from 1881. The diagram shows a series of wavy lines representing different soundings taken at different times. A dashed line at the top is labeled "Bottom of River June 1887". Below it, a solid line is labeled "TRAINOR SOUNDINGS 1881". Another solid line is labeled "JENKINS 1889". A horizontal line near the bottom is labeled "LINE OF BED ROCK". A vertical dashed line on the left is labeled "of large boulders". The bottom of the diagram shows a hatched pattern representing the river bed.

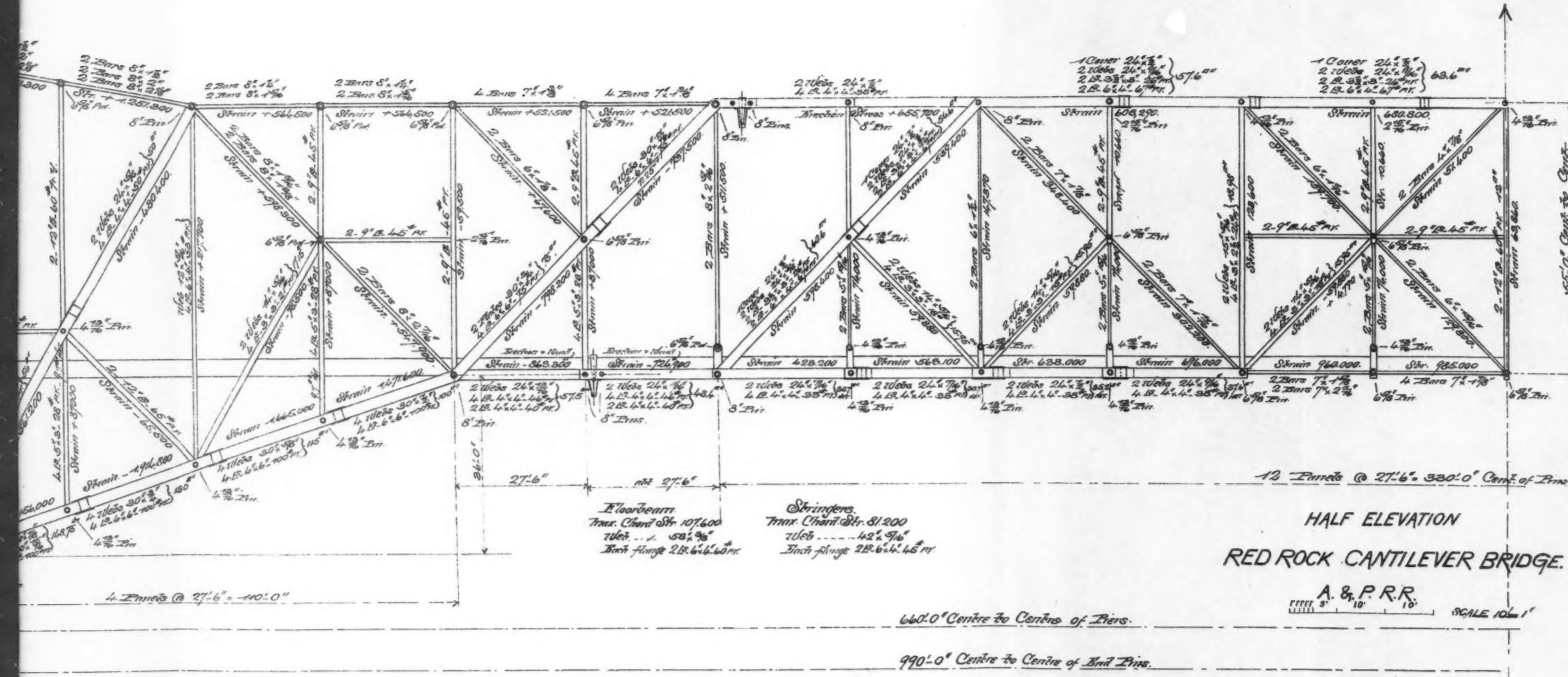
PLATE CXXIII.  
TRANS.AM.SOC.CIV.ENGRS.  
VOL.XXV. N<sup>o</sup> 518.  
RED ROCK CANTILEVER BRIDGE.

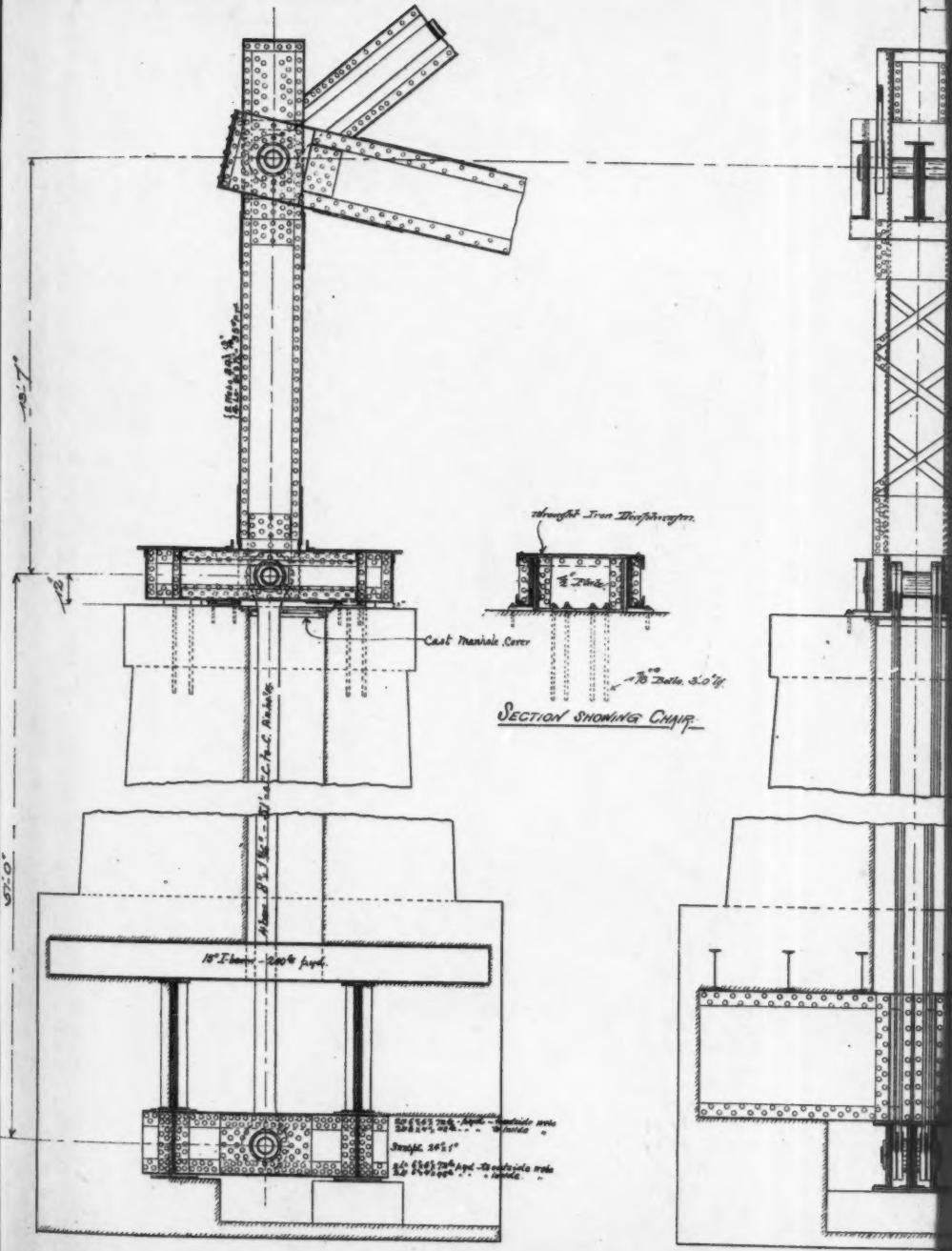


GENERAL ELEVATION AND PROFILE  
RED ROCK CANTILEVER BRIDGE  
ATLANTIC & PACIFIC RY.









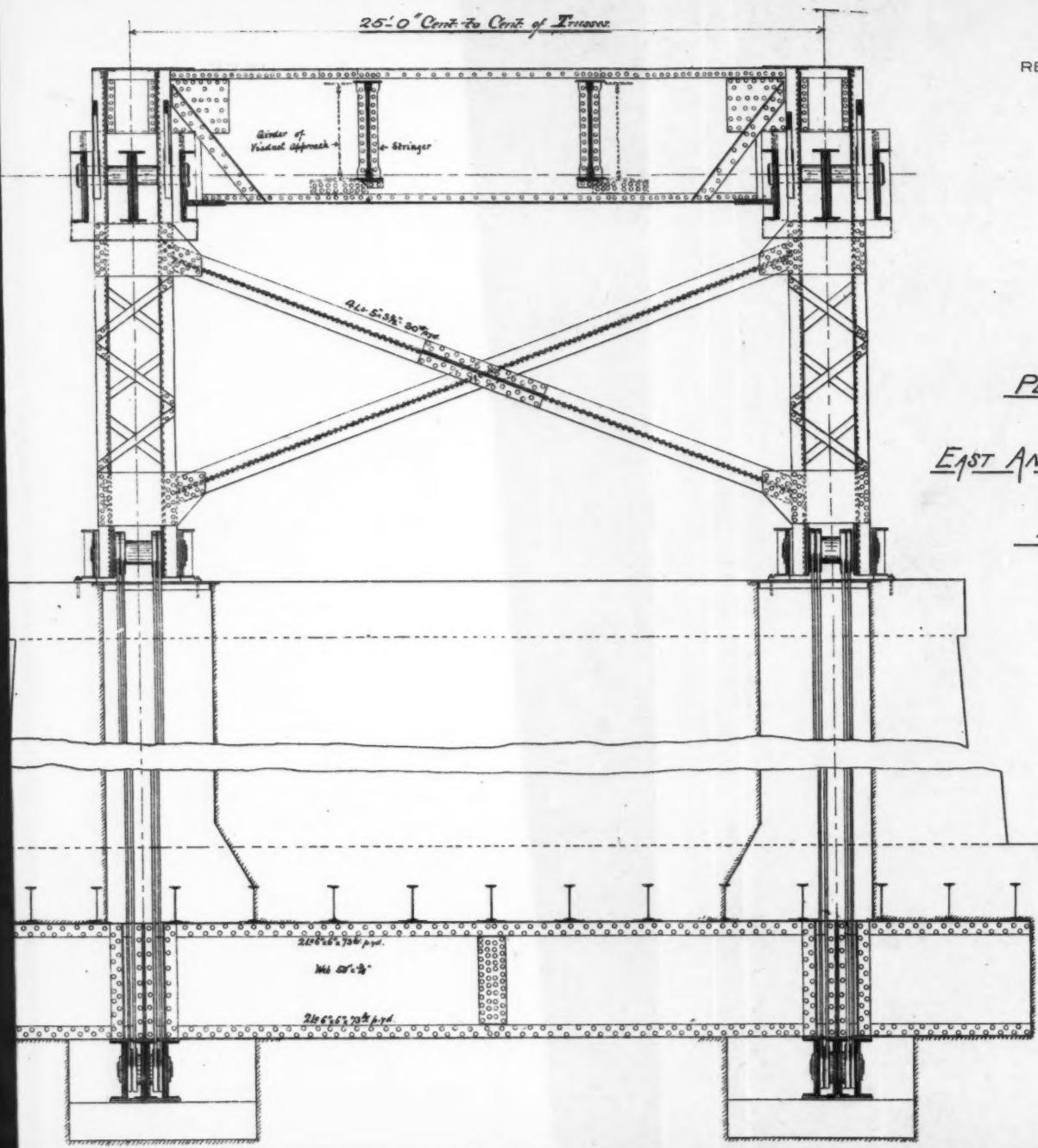
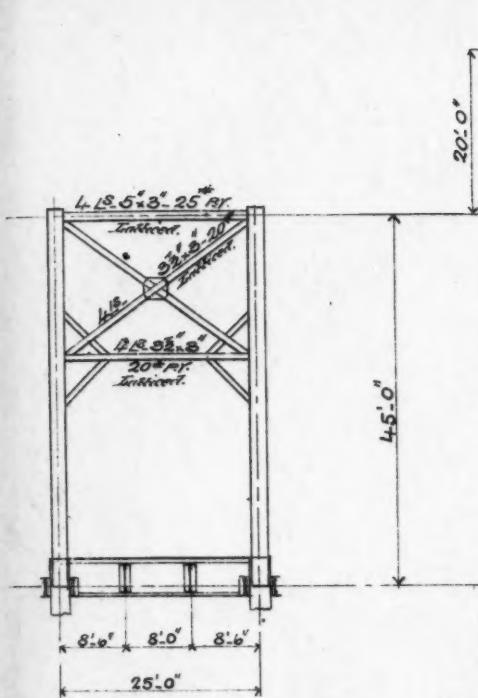


PLATE CXXV.  
TRANS.AM.SOC.CIV.ENGRS.  
VOL XXV. NO 518.  
RED ROCK CANTILEVER BRIDGE.

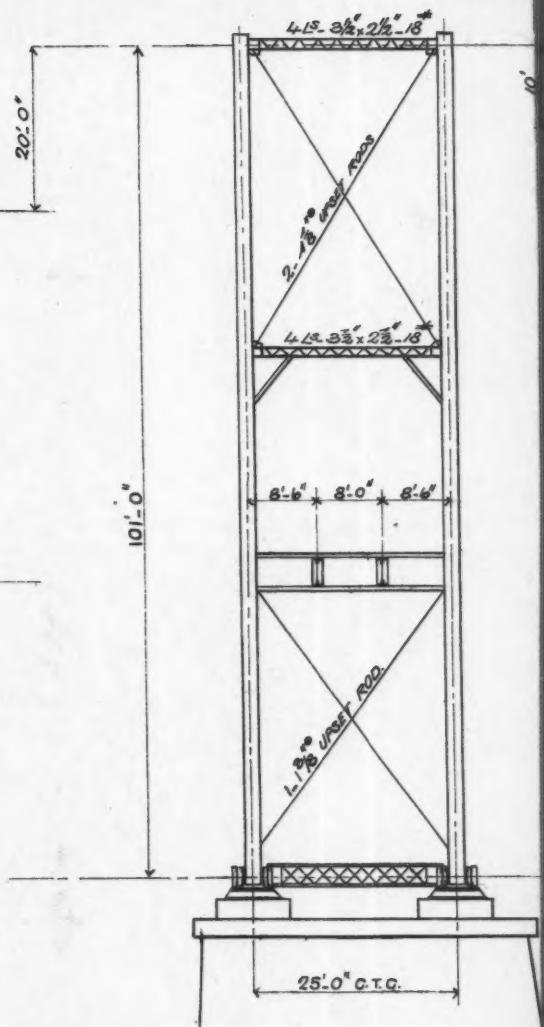
PLAN OF ANCHORAGE

EAST ANCHOR PIER - RED ROCK BRIDGE.

1 1/2" P.R.R.

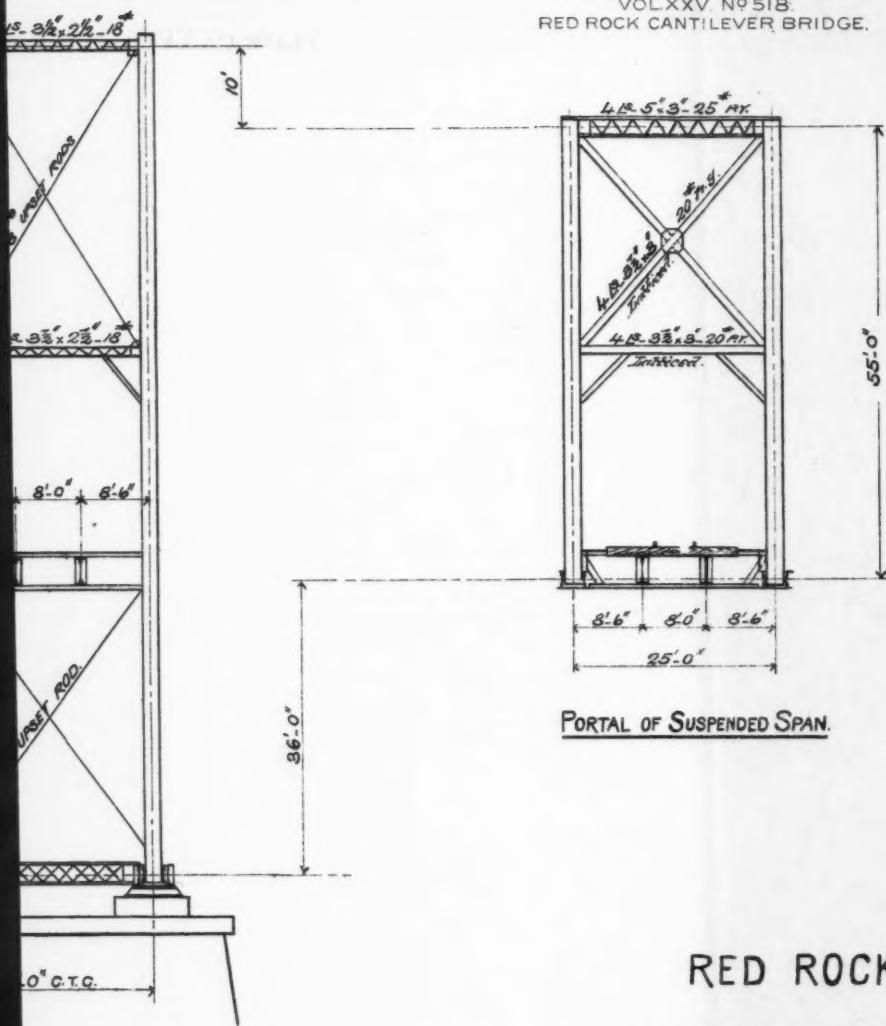


END PORTAL.



TRANSVERSE SECTION  
OVER MAIN PIER.

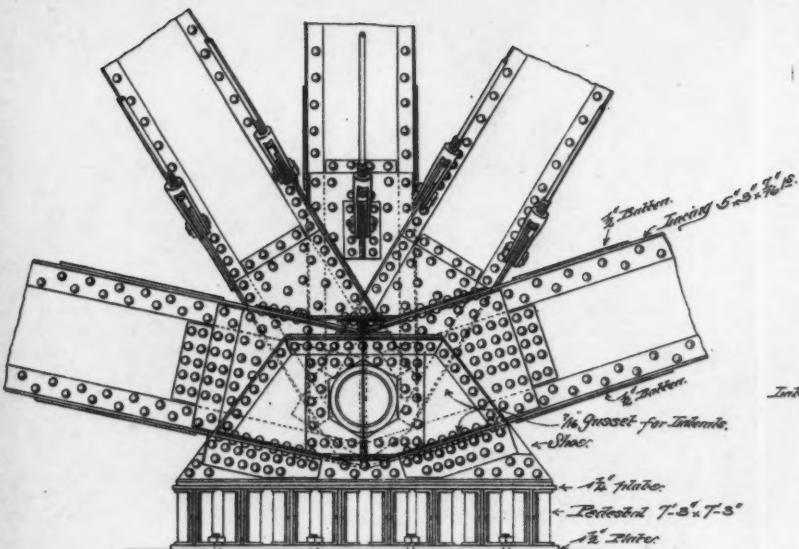
PLATE CXXVI.  
TRANS. AM. SOC. CIV. ENGRS.  
VOL. XXV. N<sup>o</sup> 518:  
RED ROCK CANTILEVER BRIDGE.



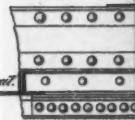
RED ROCK  
CANTILEVER BRIDGE.

A. & P. R.R.

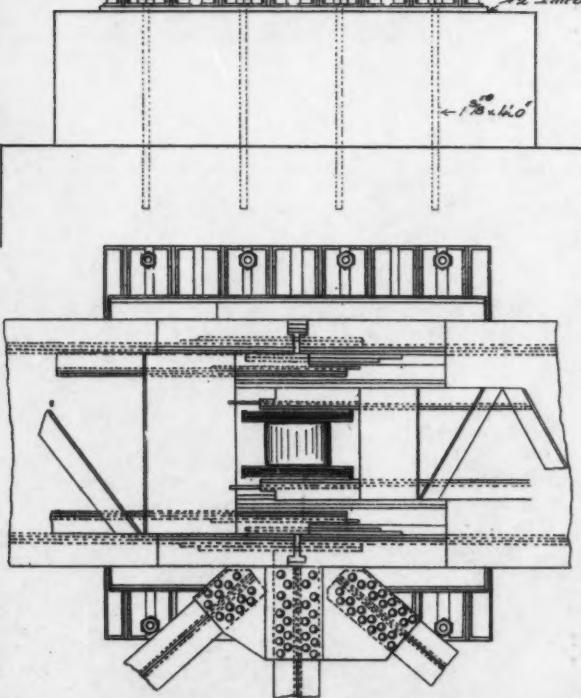




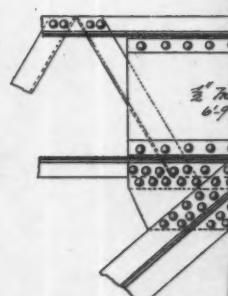
Intermittent.



III Prints 15



DETAIL OF SHOE PEDESTAL.



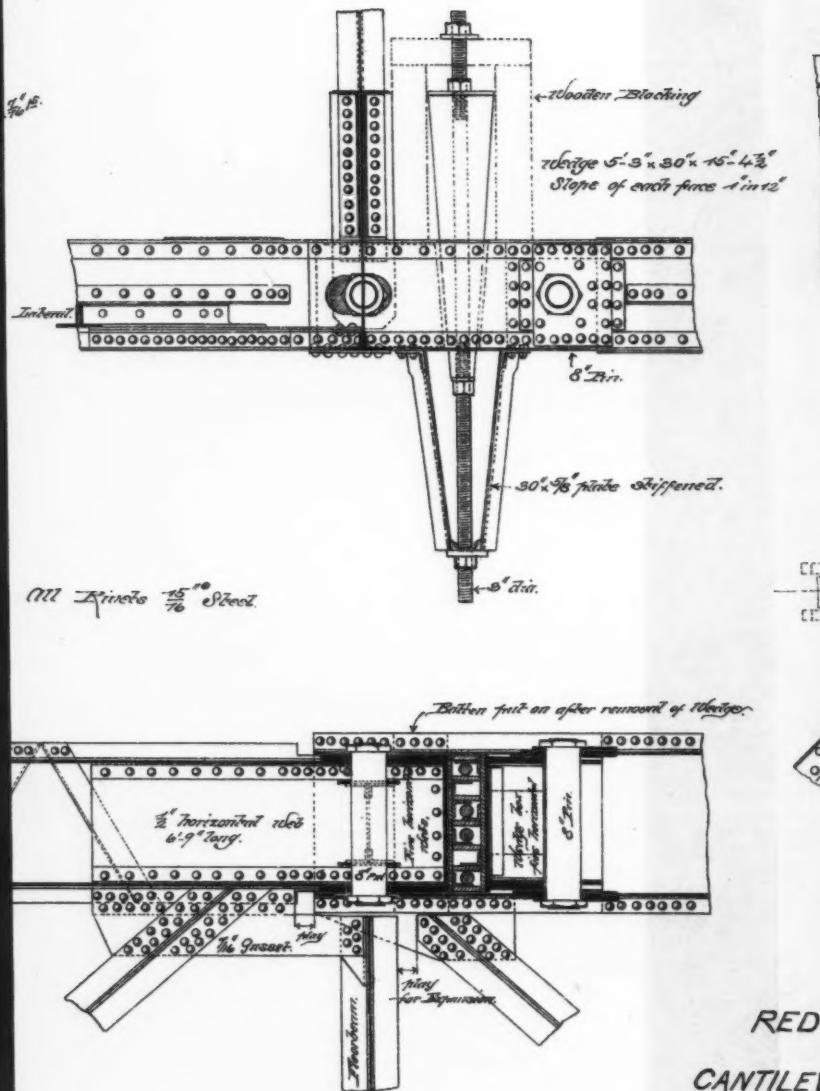
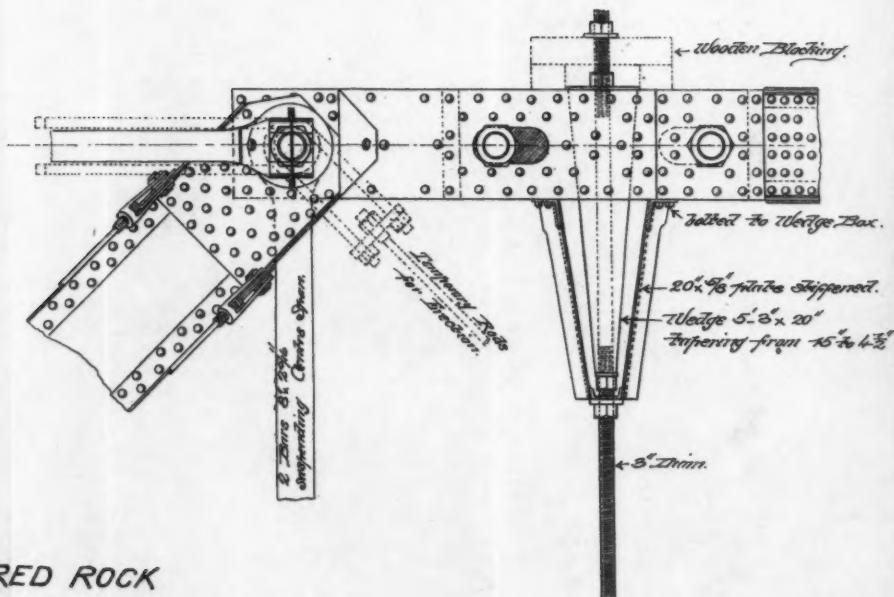
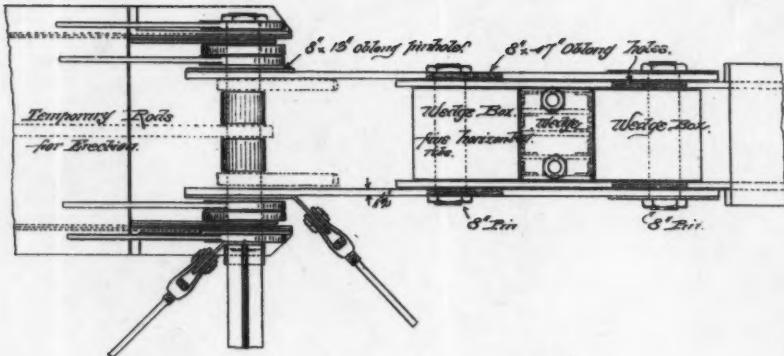


PLATE CXXVII.  
TRANS.AM.SOC.CIV.ENGRS.  
VOL.XXV. N° 518.  
RED ROCK CANTILEVER BRIDGE.



RED ROCK  
CANTILEVER BRIDGE  
A. & P.R.R.

8", 12", 18", 24"  
DETAIL OF UPPER WEDGE.



length, as fixed in setting the pedestals, was correct within a fraction of an inch. The process of withdrawing the wedges was at once begun and continued until the connecting pins were all driven and the wedges entirely loose, the chords of the center span taking their normal stresses. Some of the wedges moved up and out as fast as the lower nuts were unscrewed. Others had to be started by a light blow after the nuts had been moved a little.

The day after the connection was made and before the traveler was taken down, a washout at the old crossing of the river stopped all traffic on the road, and on the day following the washout, the track over the center was laid and cleared, and trains passed between the legs of the traveler, which had been lightened by the removal of engines, rigging and lower bracing. The structure thus carried a concentrated load of 110 tons at the center of the span in addition to the regular live load. Their mission ended, the wedges, bearing boxes and pins were removed, and the separate portions of each upper adjustment panel riveted together to form one member, its length being fixed according to the temperature at the time.

APPENDIX "A."  
RECORD OF RED ROCK CAISSON—SINKING.

DATE.	Elevation of River.	Height of Caisson.	ELEVATION OF TOP OF CAISSON AT CORNERS.			Mean Ele. of Cutting E.	POSITION.			PROGRESS.	Fee <sup>s</sup>
			Northeast.	Northwest.	Southeast.		North.	South.	East.		
Nov. 18th.....	469.80	13.00	481.60	481.68	481.66	481.66	488.56	488.50	488.50	488.50	1 <sup>1</sup> / <sub>2</sub> " N.
17th.....	469.70	15.77	484.29	484.36	484.30	484.30	488.54	488.50	488.50	488.50	0.62
19th.....	469.70	16.75	485.20	485.23	485.27	485.27	488.49	488.49	488.49	488.49	0.05
20th.....	469.60	18.73	487.22	487.21	487.24	487.24	488.49	488.49	488.49	488.49	0.00
21st.....	469.65	19.70	488.19	488.19	488.23	488.23	488.49	488.49	488.49	488.49	No Obs.
22d.....	469.45	19.70	487.39	487.39	487.27	487.34	487.69	487.69	487.69	487.69	0.00 Air. Int.
23d.....	469.57	19.70	486.13	485.93	485.97	485.97	486.28	486.28	486.28	486.28	1.41
24th.....	469.70	19.70	484.59	484.47	484.42	484.42	484.60	484.60	484.60	484.60	1.48
25th.....	469.77	21.70	484.45	484.06	484.52	484.52	484.66	484.66	484.66	484.66	2.28
26th.....	469.81	22.70	484.98	484.88	484.68	484.68	484.77	484.77	484.77	484.77	0.30
27th.....	469.90	23.70	484.72	484.72	484.49	484.49	486.83	486.83	486.83	486.83	1.30
28th.....	469.88	24.70	484.76	484.94	484.94	484.94	486.10	486.10	486.10	486.10	0.73
29th.....	469.85	27.00	487.14	487.13	487.23	487.38	489.53	489.53	489.53	489.53	0.57
30th.....	469.80	27.70	486.70	487.80	487.96	487.80	488.10	488.10	488.10	488.10	0.43
Dec. 1st.....	469.75	27.70	486.33	486.22	486.18	486.25	488.55	488.55	488.55	488.55	0.26
2d.....	469.91	27.70	485.80	485.44	485.70	485.48	487.90	487.90	487.90	487.90	0.65
3d.....	470.00	30.60	488.01	487.74	487.78	487.78	487.11	487.11	487.11	487.11	0.79
4th.....	470.50	30.60	487.09	486.90	486.59	486.56	486.18	486.18	486.18	486.18	0.33
5th.....	470.70	30.60	486.34	486.38	485.78	485.56	485.50	485.50	485.50	485.50	0.98
6th.....	470.60	31.60	486.27	486.15	486.15	486.15	486.37	486.37	486.37	486.37	0.80
7th.....	470.65	33.60	487.61	487.46	487.49	487.49	488.45	488.45	488.45	488.45	0.51
8th.....	470.70	33.60	487.08	487.03	486.97	486.97	488.39	488.39	488.39	488.39	0.87
9th.....	471.00	33.60	486.31	486.08	486.08	486.08	487.52	487.52	487.52	487.52	0.67
10th.....	471.00	33.60	485.72	485.41	485.26	485.26	485.41	485.41	485.41	485.41	0.77
11th.....	471.00	33.60	485.09	485.09	484.56	484.56	484.94	484.94	484.94	484.94	1.03
12th.....	471.00	33.60	485.90	486.11	485.13	485.13	485.05	485.05	485.05	485.05	0.96
13th.....	472.20	33.60	484.56	484.57	484.75	484.75	484.99	484.99	484.99	484.99	0.40
14th.....	472.20	33.60	484.59	484.54	484.77	484.77	484.99	484.99	484.99	484.99	0.82
15th.....	473.00	33.60	485.78	487.94	483.71	483.71	484.04	484.04	484.04	484.04	0.16
16th.....	473.00	33.60	482.78	482.78	482.78	482.78	487.11	487.11	487.11	487.11	1.16



## APPENDIX "B."

WEIGHT OF METAL IN BRIDGE, BY HENRY H. QUIMBY.

	IRON.	STEEL.	TOTAL.
East anchorage, exclusive of floor beams.....	27 400	51 035	78 435
West anchorage, exclusive of floor beams.....	27 100	65 388	92 488
Floor of anchor and cantilever arms, including 16 600 pounds of reinforcing stringers for use in erection...	112 940	158 570	271 510
Two anchor arms.....	227 260	742 610	969 870
Two cantilever arms.....	223 270	696 070	969 870
Metal over piers (vertical posts, pins, transverse rods and struts).....	65 830	112 960	178 790
Expansive panels (chords and x posts).....	18 470	109 700	128 170
Temporary members (wedges, reinforcing bars, etc.).....	61 570	14 470	76 040
Suspended span.....	399 375	302 600	701 975
Total iron, pounds.....	1 163 215		
Total steel, pounds.....		2 253 403	
Total metal, pounds.....			3 416 618

## APPENDIX "B"—(Concluded).

COMPUTATION OF PRESSURE ON PIERS, BY HENRY H. QUIMBY.

Anchor arm, including iron floor.....	620 700
Cantilever arm, including iron floor.....	659 600
Anchorage reaction (excess of cantilever over anchorage)....	30 000
Pedestal posts and bracing over pier.....	89 400
Suspended span, including anchorage reaction.....	702 000
Track, at 450 pounds per foot, including anchorage reaction.	297 000
Live load, at 3000 pounds per foot, including anchorage reaction.....	1 980 000
Total on pier.....	4 378 700
Total on one pedestal 7 feet 3 inches square.....	2 189 350

$(7 \text{ feet } 3 \text{ inches})^2 = 7569 \text{ square inches and } \frac{2189350}{7569} = 289 \text{ pounds per}$   
square inch.

## APPENDIX "C."

## COLORADO RIVER CROSSING.

Line recommended *via* Red Rock Crossing.

3 miles new rails and fastenings (60-pound rail)....	\$16 060 00
9 500 new ties at 72 cents.....	6 840 00
Laying and surfacing 13 miles track.....	10 000 00
Grading 13 miles new line.....	89 315 00
Pile and trestle bridges.....	33 900 00
Masonry and iron work at Colorado River Crossing	241 210 00
Total <i>via</i> Red Rock Crossing.....	\$397 325 00

Change of line, retaining present crossing, grading and protection of banks.....	\$75 950 00
3½ miles new rails and fastenings.....	15 736 00
11 083 new ties at 72 cents.....	7 980 00
Pile and trestle bridging.....	10 087 00
Masonry and iron work for permanent structure, with draw span 160 feet clear.....	271 335 00
Estimated cost of raising grade at Powell.....	10 000 00
Riprap at ends of bridge,.....	10 000 00
Total.....	\$404 088 00
In addition to the above, there is a present charge of \$6 000 00 per annum for demurrage, which is probably less than it would cost to rectify the channel and maintain a drawbridge. This amount capitalized at six per cent. per annum, would equal.....	100 000 00
Grand total via present crossing.....	\$604 088 00

NOTE.—To use the present wooden bridge, it would be necessary to add 500 feet to its length and to construct a permanent iron draw span on the plan contemplated in the second item of this exhibit. This will require an outlay of \$81 650 00, which, added to the cost of change of line, equals..... 214 353 00  
 If the dike is built before high water of this year, 500 feet must be added to the present bridge, at an expense of..... 17 500 00

Report signed by

A. A. ROBINSON,

Chief Engineer,

*Atchison, Topeka and Santa Fe Railroad.*

F. W. BOND,\*

Resident Engineer,

*St. Louis and San Francisco Railway.*

Report made about 15th February, 1888.

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\* F. W. Bond acting for Chief Engineer James Dun on account of his not being able to attend.

**APPENDIX "D,"**  
**CONDENSED STATEMENT OF COST, FINAL, JUNE 1ST, 1890.**

No.	Division of Accounts.	Caisson.	Masonry.	Preparatory.	Grading.	Pile Bridges.	Cross-Ties.	Rails.	Track Fastenings.	Track Laying.	Superstructure.	Total.
A..	Labor.	\$71,754.02	\$65,297.39		\$68,339.86	\$17,274.04				\$65.49	\$12,658.43	\$2,138.79
B..	Timber.	7,665.02				13,069.83						22,419.68
C..	Piling.	315.24				10,717.82						11,032.66
D..	Iron and steel.	2,180.13				2,650.91						218,268.87
E..	Tools and material.	8,415.65										625.68
F..	Fuel and water.	7,158.30				1,180.75						10,210.57
G..	Cement.	9,568.00	12,396.80									10,312.65
H..	Cross-ties.											21,944.80
I..	Rails.											16,800.00
J..	Track spikes.											30,160.00
K..	Angle plates.											1,983.72
L..	Track bolts.											2,987.46
M..	Servis tie plates.											632.34
N..	Freight A. & P.	7,641.17	8,260.00		\$295.15	1,941.18	4,092.86	3,316.50	4,460.09		4,928.90	
O..	Freight G. S. & A. T.	5,723.63				1,360.14	3,921.14	5,487.88	11,028.88			31,832.24
P..	Local and train service.	2,700.90	1,295.60				1,927.85					29,514.04
Q..	Soundings.					7,808.79						8,372.80
R..	Tramway and tracks.					6,238.17						7,868.79
S..	Demurrage.					900.00						940.00
T..	Track to quarry.	6,652.61	3,127.64			962.41	2,646.64	3,165.52				7,313.63
U..	Engineering.											24,060.33
Total .....		\$128,265.19	\$60,297.65			\$23,148.10	\$14,277.82	\$65,660.22	\$25,604.44	\$45,618.67	\$16,921.72	\$105,669.02
A. & P. Construction account No.	4.	4.	4.	3.	4.	5.	6.	7.	8.	4.	Construction.	

Total cost: Superstructure..... \$280,154.74  
 Substructure..... 282,278.94

Total..... \$462,433.68, being the sum cost columns 1, 2, 3 and 10.

The large amount (\$20,748.10) for preparatory work was due to the natural difficulties of the situation above those ordinarily encountered in such work, including the raising of material, building and maintaining tramway to caisson site, etc.

## APPENDIX "E."

## SPECIFICATIONS FOR BETON.

The beton, for filling the chamber of the caisson, crib above the chamber and the anchor piers, shall consist of English Portland cement, clean sand, gravel and broken stone, adding water (a minimum quantity), and mixed and proportioned in accordance with the directions of the engineer in charge. The broken stone shall be of rock of hard quality, broken to size of 2-inch cubes, and shall be washed clean of all dirt by throwing water over it before adding to it the cement and sand. Stone of one-half to 2 cubic feet in dimensions may be used in the caisson and piers in such manner as may be directed by the engineer in charge.

**MIXING.**—The cement and sand shall be mixed dry in convenient quantities and proper proportions by measure, and then wetted to the consistency of wetted meal, care being taken not to get too great a quantity of water; then add the broken stone, washed as mentioned before, the mass to be in place without delay, in courses not to exceed 8 inches in thickness, except where large stone are used in the structure, as in masonry, when two or more courses shall be used to fill in the course of masonry. The beton, before putting in place, shall be thoroughly mixed, either by hand with shovels or by machine adapted to that purpose, and immediately on being put in place shall be well and thoroughly rammed with iron rammers. If water is observed to rise and cover the surface of the course under the effect of the tamping, it will be taken as indicating too much water and the quantity of water will be reduced in the succeeding course. All courses shall be carried uniformly over the surface of the structure, taking care where "one man" stone are used to have no stone lay against a retaining plank, and that the space between be wide enough to secure perfect freedom in the use of a rammer.

**PROPORTIONING CEMENT IN THE BETON.**—Referring to the proportion of cement to be used, there will be three qualities of work.

**First.**—For leveling of heavy pier on rock foundations where great strength is needed, the proportion will be one part English Portland cement to two parts of sand.

**Second.**—For filling the chamber of the caisson and for the first 6 feet of the crib above roof of chamber; for that portion of anchor piers

below and 2 feet above the anchor beams, and for the foundation of pedestals and shore abutment not exceeding 2 feet in thickness, one part of cement and three of sand.

*Third.*—For the upper portion of the anchor piers, for filling the crib of the main pier (caisson crib above first 6 feet), and for all foundation work over 2 feet in thickness, one part of cement to four parts sand.

WEIGHT AND PROPORTION.—Taking No. 1, and allowing a shrinkage in volume of all the different ingredients of 20 per cent., which is about what has been found by careful tests on half-yard samples, there will be required for each cubic yard in place 27 cubic feet + 20 per cent. (= 5.4 cubic feet) = 32.4 cubic feet of material per cubic yard. Fifty-five per cent. of this (17.8 cubic feet), or 5.9 barrels (nearly), will be broken stone. Forty-five per cent. should be cement and sand = 14.6 cubic feet, divided as follows :

	Cubic feet.	Barrels of 3 cubic feet.
English Portland Cement.....	4.86	= 1.62
Sand .....	9.74	= 3.25
Broken stone.....	17.80	= 5.93
Total.....	32.40	= 10.80

No. 1. Not to be used to exceed 6 inches in depth, except when used to fill between face stones and backing in masonry.

No. 2. On the same basis (*i. e.*, 32.4 cubic feet per cubic yard), we will have as before, 17.8 cubic feet of broken stone, and 14.6 cubic feet sand and cement, divided as follows :

	Cubic feet.	Barrels of 3 cubic feet.
English Portland Cement.....	3.65	= 1.22
Sand .....	10.95	= 3.65
Broken stone.....	17.80	= 5.93
Total.....	32.40	= 10.80

No. 3. Broken stone as before, 17.8 cubic feet, and cement and sand 14.6 cubic feet, divided as follows :

	Cubic feet.	Barrels of 3 cubic feet.
English Portland Cement.....	2.92	= 0.97
Sand .....	11.68	= 3.90
Broken stone.....	17.80	= 5.93
Total.....	32.40	= 10.80

It is considered practicable to use "one man" stone to the extent of from 25 to 40 per cent. of the total volume, thus saving a considerable quantity of cement. This refers, of course, to those portions of the work where mass and weight are required. When special strength is required, only the neat beton, as above, will be used. These "one man" stone, after being placed, should be wetted with a sprinkler to secure perfect adhesion to the beton when rammed between. The surface of all face stone and the backing in masonry should also be so wetted.

An approximate and uniform measurement of all material used is imperative. A half barrel measure is good, but the cubic foot measure is preferred.

The time in which the beton should be in place should not exceed forty-five minutes; less time is much safer. Water should be sparingly used and the mixing thoroughly done. A competent inspector will be placed on the work while it is being done, who shall see that all the directions are carried out and that the tamping is thoroughly done.

When each course has been thoroughly tamped and leveled off, it shall be immediately wetted thoroughly over its whole surface, to dissolve and smooth down all crumbs of mortar and dust so as to secure perfect adhesion of the succeeding course.

In the body of the main piers, beton will be used to fill between the face stone and the backing and between the stones of the latter, care being taken that the backing stones be so placed as to allow free use of the rammer between. Then, after the face seams are well filled with mortar and the surface of all stones wetted, the beton No. 1 will be deposited and tamped, using two or more courses as the depth of the course of masonry may require, until the same shall be fully leveled up to a true surface and bed for the succeeding course, and not omitting the thorough wetting of the surface, as required on the beton work before mentioned.

Clean gravel, sifted and assorted, with a minimum diameter of one-eighth inch to a maximum diameter of 2 inches, may be substituted for the broken stone in the concrete filling of the crib of the caisson. Only coarse, clean river or pit sand will be used. No wind drifted sand or such river sand as may contain earth, mud or dust will be allowed.

NOTE.—Observation in relation to the use of water during the progress of this work has tended strongly to confirm the correctness of the conclusions drawn by Gillmore and others as to the reduction of the amount of water used in this kind of work. The writer hereof, in some experiments in 1886, found that it was difficult in hand mixing to reduce the amount used below 14 per cent. of the mass, but in the work by the machine at Red Rock this was reduced to about 10 per cent., or about double that of natural absorption by solid rock. In this case an increase in amount of water used in mixing visibly reduced the strength of the work. It may be remarked, however, that the variation between different rocks or sands will have much to do with this question of the proper amount of water.—*Rove.*

## APPENDIX "F."

## TESTS OF FULL SIZE EYE-BARS—STEEL.

SECTION.	Mark.	Ultimate Strain, Pounds per Square Inch.	Elastic Limit, Pounds per Square Inch.	Elongation.	Reduction of Area, Per cent.	REMARKS.
8 x 1 $\frac{1}{8}$ "	W	41 490	31 660	4.1 per cent. in 23 feet.	.....	Crystalline, broke in head in flaw.
Same bar, reheaded and re- annealed	.....	56 360	34 600	10.5 per cent. in 20 feet.	49.6	Silky, broke in body.
8 x 2 $\frac{1}{8}$ "	A C B	50 120	28 915	18.3 per cent. in 24 feet.	48.0	Silky, broke in body.
8 x 2"	A C C	48 345	27 570	15.8 per cent. in 24 feet.	53.7	Silky, broke in body.
7 x 1 $\frac{1}{8}$ "	C B	52 460	32 100	12.3 per cent. in 25 feet.	57.1	Silky, broke in body.
8 x 1 $\frac{1}{8}$ "	C F C	52 070	30 920	19.0 per cent. in 25 feet.	55.5	Silky, broke in body.
8 x 1 $\frac{1}{8}$ "	C D B	57 050	33 870	16.4 per cent. in 25 feet.	32.6	Silky, broke in body.
7 x 2 $\frac{1}{16}$ "	3 A	57 465	33 970	15.7 per cent. in 25 feet.	49.6	Silky, broke in body.
8 x 1 $\frac{1}{8}$ "	T O B	52 220	32 530	11.4 per cent. in 20 feet.		Fifty per cent. granular. Broke in neck, in flaw.

## DISCUSSION.

CHARLES MACDONALD, M. Am. Soc. C. E.—It is scarcely possible to discuss this paper from an engineering point of view, owing to circumstances which seem to have prevented a careful preliminary study of the problem. It is rather the record of a commercial enterprise than a professional report; and is an apt illustration of the old adage, “The more haste, the less speed.” Had a complete set of borings been made, before any contracts were entered into, the subsequent proceedings would not have been complicated by obligations hurriedly undertaken, and which have resulted in an expensive and unsatisfying solution of a comparatively simple problem in construction. As to what might have been done in the way of designing a bridge for this location, which would have fulfilled all requirements at the least cost, it is not necessary now to take account.

The bridge described in the paper before us is doubtless designed with the proper margin of strength; and, in detail of connection, follows the best practice. There are, however, certain strategic principles of construction, involving general outline and subdivision of spans, which may be considered with advantage in this connection. One of these implies that lines of beauty should, of necessity, be lines of strength, while lines of strength may not always be lines of beauty. Nature, the wisest and best constructor, has established this law; and we may assume, as a corollary to it, that the best construction is that which combines scientific proportion with artistic excellence in design. If, therefore, the general outline presents abrupt and ungraceful lines of strength, it is evident that the law of correct proportion has not been complied with; and that an unnecessary expenditure of material and consequent expense has been involved in accomplishing the desired result. An inspection of the drawings would seem to indicate that harmonious outline was not a factor in the design to any considerable extent; and, for this reason, it is fair to assume that a minimum of cost has not been reached, although the safety of the structure is assured beyond question.

There are two questions of detail, which may be referred to in closing, viz.: Concrete in piers; and, steel inspection. In climates where frost does not exercise an injurious influence, it is believed that concrete might be utilized to a much greater extent than at present. The anchor piers of this bridge were of concrete; but the main piers were of masonry, presumably much the most expensive. The writer has built several river piers of concrete throughout, with good results.

As to surface defects in steel plates, for which cause large quantities of material were rejected, much has yet to be learned in this country. The writer has frequently had occasion to test plates which had been

rejected for so-called surface defects, and found them, in almost every instance, sound and good. The entire question of inspection should receive more careful attention, in order to insure harmonious working between the producer and consumer. In this connection, Mr. Alfred E. Hunt's paper on "Tests of Structural Steel," read before the American Institute of Mining Engineers, at the October meeting, 1891, contains many valuable suggestions.

W. H. BREITHAUPt, M. Am. Soc. C. E.—With the river bottom as it was supposed to be at the site of this bridge a cantilever design was without question the proper solution. But with the actual bottom as later found, a different design would have been more economical. For two 400-foot spans and a short span, all non-continuous, the cost of masonry would have been not much in excess of the actual cost. A pier in mid-channel would probably have cost less for the same height than the present east pier, considering the difficulties encountered and the changes that had to be made during its sinking; the two main piers, for through spans with rail elevations as used, would, however, have had to be higher. On the whole, as stated, the total cost of the masonry could not much have exceeded the actual cost. The weight of two 400-foot spans and a short span to make up the length out to out of the cantilever arms would be approximately 2 400 000 pounds. The actual weight as given is 3 400 000 pounds, a difference of 1 000 000 pounds. Erection would have cost a good deal more, and would also have been attended with more risk; but as the erection of a 400-foot span is a matter of a few days only when everything is ready, such risk would have been limited to the falsework. One million pounds at six cents per pound, the price mentioned in the paper, amounts to \$60 000. Allowing, say, \$20 000 for greater cost of erection and contingencies, reduces this to \$40 000 as the difference in cost, had the survey on which the work was undertaken, established the actual conditions.

J. F. WALLACE, M. Am. Soc. C. E.—One of the most interesting features of Mr. Rowe's paper, is the account of the difficulties experienced in determining the actual line of the bed rock at this locality. This is not the first time that engineers in charge of bridge surveys have been misled by boulder beds and have considered the bed rock to be at a higher elevation than it actually was; the true facts only developing upon the sinking of the caissons for foundations. A recent example of this kind was the Chicago, Milwaukee and St. Paul Railroad bridge over the Missouri River at Randolph, where the boulder bed was misjudged to be the solid rock. The original soundings at Sibley for the Atchison, Topeka and Santa Fé bridge over the Missouri River at that point, developed a boulder reef, which was at first supposed to be bed rock, but the soundings taken immediately before the construction of the bridge penetrated the various boulder strata and developed the true location of the bed rock before the substructure plans were made. It is not sur-

prising that the borings made at the Red Rock bridge in 1881, should have so misled the engineer, as the top of the boulder was at such a uniform level, that he was easily deceived in supposing the rock encountered to be the bed rock; but it is surprising that the second series of soundings taken in 1888, should have failed to disclose the fact that the east side of the bed of the river was composed of boulders and drift and should have failed to discover the true line of the bed rock.

In taking borings for an extensive and important structure, or, in fact, any structure that renders the taking of borings necessary to secure suitable foundations, the engineer in charge should carefully study the geological formation of the region in which it is to be erected, and the surroundings and conditions of the immediate locality. The borings should be carried to such a depth as to actually determine the character of the strata. In this case it does not seem that it would have been difficult to have determined the character of the rock encountered by the drill. The history of this bridge should certainly be a lesson, not only to engineers, but also to projectors and investors having control of enterprises of this kind. It is a common thing for capitalists to consider the money spent in engineering, particularly on the preparatory work, as an expense to be avoided, if possible, and in a large measure as money wasted; and this is a striking instance of where rush and hurry prevented proper examinations that would have saved a year's time and thousands of dollars to the corporation interested. One interesting item in Appendix "D" is the cost of the various soundings, which is noted as \$7 808.79, a sum certainly large enough to have obtained accurate information as to the location and character of the bed rock, if it had been expended at the proper time and in the proper manner. The record from Mr. Rowe's paper would seem to indicate that the tools and appliances furnished for these borings were inadequate, and this fact seems to have been one of the causes that added to the expense of the borings and also to the unsatisfactory results obtained in this connection. The writer would like to mention a few facts in regard to the soundings taken for the Sibley bridge. Some of the soundings passed through several distinct boulders.

A large casing pipe was used with a steel shoe, and after drilling through the first boulder encountered, a smaller casing pipe was inserted, and by proceeding in this way, it was possible to drill through several boulders and through sand and gravel underneath. After the bed rock was reached, one boring was taken to the depth of 165 feet to determine accurately the strata at this point. Several other borings were taken far enough into the strata to determine accurately its character. This gave assurance that the intermediate borings would indicate the true elevation of the bed rock. The cuttings from the boring tools were carefully preserved and labeled and the location from which they were obtained accurately determined, so that the record shown was very com-

plete and was afterward verified in the sinking of several caissons. One interesting feature that was developed in the sinking of the caissons was the discovery of different boulders that had been encountered by the boring tools. Several boulders were discovered that had been entirely perforated by the drill and other smaller boulders were found that had been turned by the drill. One boulder was found about 12 inches in diameter that showed in two places where the drill had entered it for about 2 or 3 inches, and the drill had then turned the boulder partly in a gravel bed. The drill had then been removed and reset, when it struck the boulder nearer the center and passed entirely through it. Mr. Otto Sonne, M. Am. Soc. C. E., had the immediate charge of the party engaged in these borings, which were made under the direction of the writer as Resident Engineer, and O. Chanute, Consulting Engineer. The borings at Sibley covered the entire bridge site and were taken approximately at the four corners of the probable location of each pier, and also extended across the entire valley at intervals of about 1 mile.

The valley at this point was from 8 to 10 miles wide. The geological formations were, therefore, fully determined and no chances were taken. The writer does not remember the exact figures in regard to the cost of the borings at Sibley, as they were included in the general engineering account, but their total cost was approximately \$5 000. Their accuracy contributed largely to the successful and economical prosecution of the work.

EDWIN THACHER, M. Am. Soc. C. E.—The results of the investigation by S. W. Robinson, M. Am. Soc. C. E., to determine the most economic length for the center suspended span under different specifications, are interesting and instructive, as is also his investigation of the lateral stability of the cantilever as compared with a simple span of the same length; but it is not clear to me, that because a simple span of a certain length and width has the same lateral deflection as a cantilever span of the same length and less width, the two have necessarily equal lateral stability. I have never considered lateral deflection a matter of any great consequence, as it will take care of itself; but have regarded as more important questions, the condition of the compressed chords when considered as trussed columns, and the resistance offered, to overturning by the wind. Considered as a trussed column the cantilever admits of a much less width for a given span than the simple truss; but the reverse is often true as regards overturning, and I think that this latter consideration should govern in determining the width between trusses in cantilever bridges. As the author does not consider this question in his paper, the following calculation may be of interest. The surface exposed to the wind has been taken as twice the projected surface of one truss plus the floor surface, when the structure is unloaded, and at 9 square feet per lineal foot in addition, when loaded with empty cars

weighing 900 pounds per linear foot—1 square foot per linear foot of leeward truss being covered by the train. The weight of structure has been taken from the author's first Appendix B.

## DEAD LOAD ON THE PIER:

	Pounds.
One anchor arm.....	484 935
One cantilever arm.....	459 670
Floor for anchor and cantilever arms.....	127 455
Suspension span, including anchor reaction .....	701 975
One-half excess of anchor arm over cantilever arm.....	12 632
Pedestal posts and bracing over piers.....	89 395
Track, including anchor reaction.....	297 000

Total dead load on one pier..... 2 173 062  
 " " " one pedestal..... 1 086 531

Moment of stability, bridge unloaded =  $1 086 531 \times 25 = 27 163 275$  foot-pounds.

## OVERTURNING MOMENT, WITH A WIND OF 30 POUNDS PER SQUARE FOOT:

	Foot-pounds.
One-half center span.....	$3 467 \times 30 \times 58.2 = 6 053 382$
Reaction at anchor.....	$3 467 \times 30 \times 36.0 = 3 744 360$
Anchor and cantilever arms.....	$8 352 \times 30 \times 44.9 = 11 250 144$
Total overturning moment of the bridge un-loaded.....	= 21 047 886

## FORCE REQUIRED TO OVERTURN THE UNLOADED BRIDGE:

$$\frac{27 163 275 \times 30}{21 047 886} = 38.7 \text{ pounds per square foot.}$$

When the bridge is loaded with empty cars, the moment of stability will be as follows:

	Foot-pounds.
From dead load (as above).....	= 27 163 275
From train $450 \times 660 \times 25.0$ .....	= 7 425 000

Total amount of stability, bridge loaded..... = 34 588 275

And the overturning moment will be as follows:

	Foot-pounds
From the unloaded bridge (as above).....	= 21 047 886
From a train on the center span = $270 \times 165 \times 48.3 = 2 151 765$	
From reaction at anchorage..... = $270 \times 165 \times 36.0 = 1 603 800$	
From a train on both arms..... = $270 \times 330 \times 48.3 = 4 303 530$	

Total overturning moment, bridge loaded .. = 29 106 981

## FORCE REQUIRED TO OVERTURN THE LOADED BRIDGE:

$$\frac{34 588 275 \times 30}{29 106 981} = 35.6 \text{ pounds per square foot.}$$

It is seen, therefore, that the wind specified is not sufficient to overturn the bridge. The anchor bolts also offer some resistance to overturning, so the width selected appears to be sufficient.

The effect of pin friction on the anchor bars appears to be important in a cantilever design. A curious effect of pin friction was observed in a bridge on the L. & N. R. R., built in 1867, and removed in 1891. The center pin in top chord rotated, making a complete revolution about once in three months, or about one hundred revolutions during the life of the bridge.

S. M. ROWE, M. Am. Soc. C. E.—Charles McDonald, M. Am. Soc. C. E., says: "It is rather the record of a commercial enterprise than a professional report." I presume he means the portion submitted by myself (as I can hardly conceive it applicable to the contribution by Professor Robinson and Mr. Quimby), and I assume such to be the case. Relative to his further criticisms, I agree with him in the main, but believe still that there may be exceptions—for instance, the "Forth Bridge"—and in relation to the use of concrete in the main piers, I would question its safety when, in addition to the weight of the structure, we have the wind stress on the whole structure, with an additional and dangerous element, "Torsion," acting on those piers.

Referring to the criticism of Mr. Breithaupt as to the cost of the two types of structure, I will only say that the result of our computations, made on the ground, differ from his; but to go through the whole matter would unduly extend this discussion, and may better be done in a separate paper.

J. F. Wallace, M. Am. Soc. C. E., says: "The engineer in charge should carefully study the geological formation of the region." To fully understand the situation at Red Rock bridge it is only necessary to repeat what I have already said. "The rock forming the bed rock of the river is volcanic." This, of course, settles any question of certainty, and the existence of any formation at one point gives no evidence of its extent or position.

The Colorado River is, perhaps, one of the worst rivers to deal with in the United States, and there are many points that may escape the attention of those unacquainted with it.

S. W. ROBINSON, M. Am. Soc. C. E.—The results of calculation by Edwin Thacher of the overturning moment of wind and the opposing moment of stability are certainly reassuring, and his conclusion of safe conditions is gratifying. In comparing his figures for the pressure on the main pier with those in "Appendix B (continued)," there appears to be some omission—perhaps the "expansion panel" and reaction, whereby the resulting moment of stability found by Mr. Thacher is about 10 per cent. too small. Further, taking account of the eight anchor bolts to each pedestal on the main pier, we gain approximately another 10 per cent. of stability. Adding these, we obtain the amount

of stability, 32 600 000, to counteract the wind moment of 21 050 000; or the equivalent of a wind pressure of 46 pounds per square foot. Again, for the loaded structure the 20 per cent. above should be added to the 34 588 275, making 40 000 000, to oppose the 29 107 000 wind movement; or the equivalent of 41 pounds per square foot of wind pressure.

As a further consideration to favor the stability of the bridge against overturning by wind, there can be no question but that the leeward truss will be to some extent shielded by the windward one; while 30 pounds per square foot will be conceded a fairly high figure for direct wind pressure. Just how much to deduct for the off truss may be questioned, but probably for this structure a fifth at least, whereby altogether the structure is found capable of resisting a wind pressure of 50 pounds per square foot.

Relative to pin friction, in the interesting case cited by Mr. Thacher, of a center pin in a top chord rotating one hundred times in twenty-four years, it is likely that the pin was not at all times under severe bearing pressure, from the fact that it was a center pin, and that rotation was due to the working of the parts across the slack in the holes, or possibly the jarring at a certain time during the passage of trains. Those familiar with the necessity of lubrication in machinery, of iron upon iron bearings, will hardly be willing to admit that a pin in a close fitting hole under severe pressure, as in anchor and suspension link pins, can be made to rotate one hundred times, dry, without stiffening to practical solidity by abrasion and cold welding. These conditions are quite different from those for the pin cited by Mr. Thacher. Suspension and anchor link and pin joints have been admitted into existing cantilever bridges under conditions which are believed to be positively dangerous, without lubrication, beyond a service of ten years. This is for short links, where a stiffened joint would strain parts beyond the elastic limit.

It is especially unfortunate that doubt should be thus thrown upon such vital parts as those here considered, because a failure could not do less than result in serious disaster. Hence these parts were deemed worthy of especial consideration and provision in the present structure. It is important that data on this subject from existing bridges should be collected and reported to the Society, giving examples, and particulars of deportment of the same in service.

H. H. QUIMBY, M. Am. Soc. C. E.—Mr. Macdonald's theory that artistic excellence necessarily accompanies the other virtues in a bridge is true, because such an object is judged by utilitarian principles, and what seems good is generally pleasing. The outlines of this bridge appear ungraceful because of the disproportion between the openings, a condition that was given, not designed. The members—which constitute the lines—are disposed so as to carry the stresses by the shortest roads to the piers, except where the trusses are deepened 10 feet over the main piers, the reason for which is given in the paper. Still greater

depth would have made it more imposing, but would have called for increased width. Curved chord lines would necessarily be made of a few straight panel chords, and would neither be beautiful nor compensate for the labor entailed. A better appearance to the miscellaneous eye might have been made by shortening the suspended span to 220 feet, making the lower boom of the cantilever arm similar in length and slope to the anchor arm, but it would have been at the expense of much additional metal in arms and anchorage.

Mr. Breithaupt presents an instructive comparison of what was, with what might have been done. His reference to the risk attending the use of falsework calls attention to a factor which must be recognized in estimating the cost of such work. As risk is a commodity with a more or less fixed market value, a proper percentage added to it will in some cases determine the adoption of a cantilever design, which experience teaches is safe and comparatively inexpensive to erect.

